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Erosion Control and Environment Restoration Plan Development: Matagorda County, Texas

Phase 2: Preliminary Design

James Rosati III, Ashley E. Frey, and Robert C. Thomas   August 2013

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Report 2 of a series

Abstract

A two-part study was conducted to identify structural methods to reduce beach erosion in Matagorda County, Texas, at Sargent Beach and along Matagorda Peninsula, east of the Mouth of the Colorado River (MCR). Phase 1 (Thomas and Dunkin 2012) investigated the coastal processes of the region and introduced several structural alternatives to reduce erosion at both locations. Preferred alternatives included a groin field with beach fill east of MCR and segmented breakwaters to protect Sargent Beach.

This report explains further coastal processes with a short-term, recent sediment budget, describes recent studies, provides higher-resolution numerical modeling, and includes preliminary design of the selected alternatives. Numerical models were applied to evaluate structure performance and to prepare for preliminary design of selected structural alternatives. Calibration for one of those models, GenCade, was updated based on model improvements and additional data. A Coastal Modeling System (CMS) numerical model was developed and validated against water level data.

Near MCR, the preferred alternative included three groins 1,800 ft apart with optional levels of beach fill. Several breakwater alternatives were analyzed at Sargent Beach. Complex physical processes and potential risk associated with segmented breakwaters led to the recommendation of a demonstration project consisting of a set of 10 breakwater segments northeast of Mitchell's Cut. Design and construction of additional segments will depend on demonstration project results. Preliminary design and feasibility analysis was conducted for each of the selected alternatives. A field monitoring plan for both sites is suggested to reduce uncertainty with future design of shore protection.

Contents

Report Documentation Page

Figures and Tables

Figures

Tables

Preface

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
miles (nautical)	1,852	meters
miles (US statute)	1,609.347	meters
square feet	0.09290304	square meters
square miles	2.589998 E+06	square meters
knots	0.5144444	meters per second
miles per hour	0.44704	meters per second
pounds (mass)	0.45359237	kilograms
tons (2,000 lb, mass)	907.1847	kilograms

1   Background

1.1   Purpose

The Port of Bay City, Texas, asked the US Army Corps of Engineers (USACE) to develop potential structural solutions to reduce erosion of critical beach habitat and to increase protection from tropical storms in Matagorda County. The two primary areas of concern are Sargent Beach and 2 to 3 miles of beach on Matagorda Peninsula located about a mile east of the Mouth of the Colorado River (MCR) (Figure 1). Phase 1 of the investigation of coastal processes in Matagorda County was conducted to determine which structural solutions warranted further evaluation.

Figure 1. Map of project area and region.
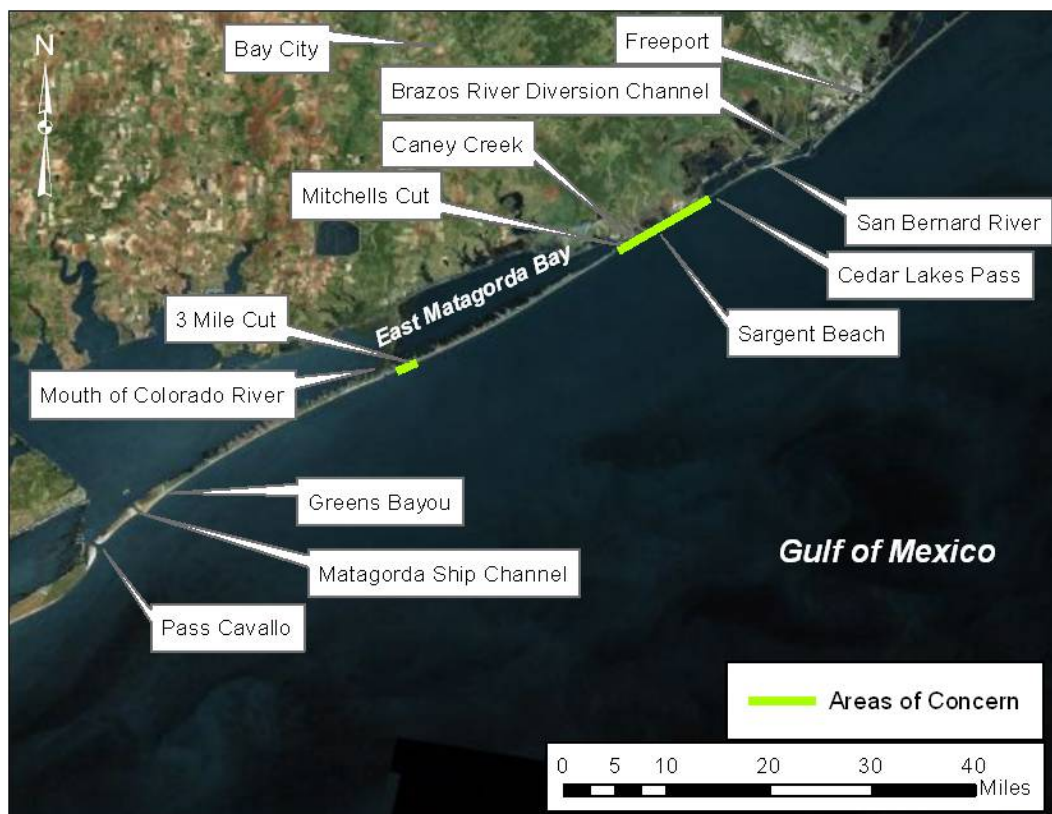
The key processes for design highlighted in Phase I:

- Net transport at Sargent Beach is small relative to gross transport.
- A strong westward net transport is located at MCR.

- Extreme erosion at Sargent Beach is a function of (a) cohesive sediment transport, which erodes and disperses fine sediment from the beach and out of the nearshore littoral system; (b) cross-shore transport, which also removes sand and fine sediment from the nearshore and beach; and (c) sediment impoundment at inlets, which sequesters sand from the Sargent Beach area. The first two processes are not easily quantified with existing technology and available funding.
- There is not enough sand in the system to maintain regional shoreline position.
- Bypassing at MCR is critical to shoreline stability and for reducing impoundment east of the jetties.

Phase 1 concluded a groin system likely would be the best alternative at MCR because it would help advance the sandy dry beach as desired. Beach maintenance will be required to maintain useable dry beach width.

For Sargent Beach, Phase I found a breakwater system would be the best way to address cross-shore transport of cohesive sediments. Performance of shore parallel breakwaters is sensitive to site conditions. Further, breakwater systems are not commonly employed on the Texas Gulf Coast, and adjacent beaches could be deprived of sediment as a result of construction. Therefore, an adaptive approach is recommended, meaning the breakwater system will be adapted to the site so adjacent impacts are minimized.

This report documents Phase 2 of this study. Phase 1 is documented in Thomas and Dunkin (2012). The terms Thomas and Dunkin and Phase 1 are used interchangeably throughout this report. Refer to the Phase 1 report for in-depth background and site characteristics.

1.2   Relevant previous studies

The Phase 1 report summarizes other studies relevant to the background of the Phase 1 investigation. Some key aspects from previous studies in modeling are discussed here.

1.2.1 San Bernard River

Sanchez and Parchure (2001) modeled the area around the San Bernard River (SBR) Mouth, including the Gulf Intracoastal Waterway (GIWW), in support of the decision to provide a new outlet for the San Bernard River. In 1999, they collected water level, current, and sediment data. The RMA2-

WES model was applied using these data and tide data provided by NOAA, as well as US Geological Survey (USGS)-provided river discharge data. Relevant items derived from the report and its recommendations were:

- The model reasonably reproduced trends in water-surface elevation.
- Knowledge of Brazos River floodgate status is essential to fully understanding water flows.
- Knowledge of marshland and shallow lakes bathymetry is necessary to reproduce the amount of water storage accurately.
- The flows are described as complex, and the importance of wind is acknowledged in the report, but winds are not mentioned in the boundary conditions used for the model.
- Collected sediment data are consistent with the sediments collected as a part of this project (Appendix C).

Kraus and Lin (2002) synthesized sediment transport processes at the SBR Mouth and applied the DYNLET numerical model to recommend a river-opening solution. The authors noted it is more difficult to accurately model a river input flooding condition, rather than a dry condition, because information such as the rate of rainfall and the topography of the flooded area are required in addition to the river discharge input.

Kraus et al. (2006) applied the ADCIRC numerical model in support of the Matagorda Ship Channel. Good agreement with measured data was obtained, and the importance of wind input was noted. Caney Creek discharge data added to the model was assigned a value of 10 % of the measured SBR discharge based on a comparison of the corresponding watershed and drainage area.

1.2.2 Experience and performance: Projects in the northern Gulf of Mexico

Edwards and Namikas (2011) and Batten et al. (2004) reviewed the work of Gravens and Rosati (1994) and reviewed the response of the beach after detached breakwaters were constructed in Grand Isle, Louisiana. Grand Isle has a wave climate, fine sediments, and shallow beach slopes similar to Sargent Beach. The GENESIS model (Hanson and Kraus 1989), a precursor to the GenCade model (Frey et al. 2012) used in this study, was used to predict the shoreline's response to a modeled set of seven detached breakwaters. Edwards and Namikas (2011) noted the shoreline shape responded in a manner similar to the one predicted, but in some areas of

the project too much shoreline recession had occurred, increasing the shoreline-to-breakwater distance and making the breakwaters ineffective at reducing wave energy at the beach. Batten et al. (2004) attributed this additional erosion to tidal currents from the nearby Barataria Pass that were affecting longshore transport in the lee of the structures, hindering the formation of salients.

Important points from Mann and Thomson (2003), Mann et al. (2004), and Campbell et al. (2005) about a separate breakwater project at Holly Beach, Louisiana:

- Depth limited waves and rock size were calculated at 1 to 3 tons.
- 315 yd^3 of beach fill were placed per breakwater segment.
- Marine mattresses decreased construction time and assured quality.
- It supported work by Dean (2001), providing a relationship to help predict shoreline response and to justify structural modifications and rehabilitation.
- It performed as designed: trapping approximately 50% of the longshore transport.
- Long-term erosive trends still applied. The resulting sediment deficit could be made up only by beach nourishment.

1.3   Report organization

This report has seven chapters. This chapter has provided background information. The others:

- Chapter 2: relevant coastal processes governing the work, including an updated conceptual sediment budget.
- Chapter 3: GenCade and Coastal Modeling System (CMS) (Lin et al. 2008) setup, calibration, and validation.
- Chapter 4: Analysis and ranking of alternatives for MCR.
- Chapter 5: Sargent Beach breakwater alternatives.
- Chapter 6: project implementation as a phased approach.
- Chapter 7: summary and recommendations.
- Appendices: numerical validation, calibration, and detailed alternative analysis (A and B); collected sand sample data (C); preliminary plans and typical cross sections for recommended alternatives (D).

2   Coastal Processes

2.1   Sediment budget update

This section describes a sediment budget for a more recent time period than analyzed in Phase 1. The GenCade model was calibrated for the 1995 to 2000 time frame in Thomas and Dunkin (2012). Additional information was collected after Phase I was finished. The 1995-2000 time frame was recalibrated with this information. Then the GenCade model was validated for the 1991 to 1995 and 1991 to 2000 periods.

2.1.1   Sediment budget

A conceptual sediment budget was developed for the region from Freeport to Matagorda Ship Channel (MSC), to provide a framework to evaluate potential alternatives for reducing erosion at Matagorda County beaches. A sediment budget is an accounting for sediments in a system, represented graphically by a series of connected cells and fluxes. Cells are reaches of the study area that are either morphologically similar (e.g., an ebb tidal shoal) or separate defined engineering actions (e.g., beach nourishment along a portion of a region, navigation channel). Sources and sinks are fluxes that provide and remove sediment from the cell, respectively. The difference between sources and sinks must equal the rate of change of volumes of sediment in each cell with consideration of the engineering activities in that cell. For a balanced cell, the sum of elements within a cell results in a residual equal to zero. In Phase 1, the analysis of the sediment budget included historical shoreline change. This sediment budget for Phase 2 will focus on short-term shoreline change from 1991 to 2011.

Data from multiple sources were applied to develop this representative historical sediment budget, expressed in terms of annual rates. The Sediment Budget Analysis System (SBAS) (Rosati and Kraus 1999) was applied for sediment budget calculations. The same sediment cells, river sand supply, and dredging quantities used in Phase 1 were applied.

2.1.2   Beach volume change

Change in beach volume for each cell was calculated based on the average short-term shoreline change rate (BEG 2011). During the Phase 1 study, recent beach profile data at Quintana, south of Freeport, and Sargent Beach were available. Additional surveys of Sargent Beach and Matagorda near

MCR allowed for the calculation of improved conversion factors. The beach profile data at Quintana, Sargent Beach, and Matagorda were translated 1 ft landward. Then, the volumetric difference between the original and translated profile was calculated to determine a conversion factor to relate shoreline retreat/advance to modern volume change. This resulted in a conversion factor of 0.9 yd³/ft² for Freeport to Brazos Beach, a revised conversion factor of 0.65 yd³/ft² at Sargent Beach based on recent surveys, and 0.7 yd³/ft² at Matagorda Peninsula. Beach profile data for other locations were not readily available. A factor of 0.8 yd³/ft² was applied at these locations. Uncertainty and seasonal and annual variation in this conversion factor introduces uncertainty directly into the conceptual sediment budget. Table 1 lists the average annual volume change in each cell for the sediment budget.

Table 1. Annual volume change (ΔV) in each beach cell based on short-term shoreline change rates calculated by BEG (2011).

Cell	Average Shoreline Change Rate (ft/yr)	Cell Length (ft)	Conversion Factor (cu yd³/ft²)	Annual Volume Change (ΔV) (yd³/yr)
Freeport to Brazos Beach	-1.4	30,500	0.9	-39,430
Brazos to SBR	4.5	15,000	0.8	54,000
SBR to Cedar Lakes	16.2	21,500	0.8	276,920
West of Cedar Lakes	-17.3	12,000	0.8	-166,080
Sargent: East of FM 457	-16.6	22,500	0.65	-242,775
Sargent: West of FM 457	-18.4	14,000	0.65	-167,440
West of Mitchell's Cut	-10.9	59,000	0.8	-514,480
East of MCR	-0.5	56,500	0.7	-19,775
MCR to MSC: North	-1.8	85,500	0.8	-123,120
MCR to MSC: South	11.4	39,000	0.8	355,680

In addition to the improvements to some of the conversion factors used in the initial conceptual sediment budget, the average shoreline change rates were calculated in the Digital Shoreline Analysis System (DSAS) (Thieler et al. 2009). The average shoreline change rates in Phase 1 represented long-term shoreline change. However, Phase 2 aims to understand the short-term, more recent period. Shoreline change rates were calculated for several periods for each of the cells shown in Table 1. The overall period of interest for this sediment budget was from 1991 to 2011. However, some of the GenCade model runs investigated a different period, so the average

shoreline change rates also were calculated for 1991 to 1995, 1995 to 2000, 1991 to 2000, 1995 to 2011, and 2000 to 2011. The data for the 1991 and 2011 shorelines did not exist for the entire reach from Freeport to Matagorda Ship Channel, so alternative periods were needed to complete the average shoreline change rate in some of the cells. For example, the 2011 shoreline was not measured south of MCR. Therefore, the average shoreline change rate for 1991 to 2000 was used instead.

Some shoreline change rates are drastically different from the historical rates of shoreline change. This resulted in different residuals when the sediment budget was entered in SBAS.

2.1.3 Cross-shore transport

Cross-shore transport is a sink for sediments in the region. Tropical storms and cold-front passages are the primary forces of cross-shore transport, although sediments might be transported beyond the typical depth of closure at inlets, or trapped in Cedar Lakes or Matagorda Bay. No rates for cross-shore sediment transport are included in the budget, although it is recognized that this is a large potential sink.

2.1.4 Relative sea level rise (RSLR)

Relative sea level rise (RSLR) accounts for 1.2 ft of shoreline retreat per year based on average beach characteristics, measured RSLR at two NOAA stations (Freeport and Rockport, Texas), and application of the Bruun rule (Thomas and Dunkin 2012). Acceleration of RSLR could cause increased shoreline retreat and beach volume change.

2.1.5 Sediment budget summary and conclusions

The preceding data represent the preliminary sediment budget processed in SBAS. Analysis revealed a non-zero residual in most cells (i.e., sediment fluxes did not match observed erosion or accretion, taking into account engineering activities such as dredging and placement). Although some of the conversion factors were refined based on recent survey data, it might be that the factor for converting shoreline change rates to volumetric change rates was not accurate for some locations. Table 2 lists the residual by cell. The cells are displayed in Appendix B of the Phase 1 report. A positive residual indicates a volume of sediment that must be lost from the cell to match the observed volume change rate. Uncertainty and normal seasonal and annual variation in input to the sediment budget also are reported in the residuals.

Table 2. Sediment budget cell residuals.

Cell	Residual (yd³/yr)
Freeport to Brazos Beach	-14,570
Brazos to SBR	-69,000
SBR to Cedar Lakes	-52,920
West of Cedar Lakes	176,080
Sargent: East of FM 457	242,775
Sargent: West of FM 457	147,440
West of Mitchell's Cut	364,480
East of MCR	19,775
MCR to MSC: North	315,120
MCR to MSC: South	-173,000
Macro-budget	2,168,180

Positive values indicate greater net loss than currently budgeted for.

One likely source of uncertainty is the broad assumption that average shoreline change, both in space and time, accurately represents beach volume change. The improved conversion factors helped reduce the uncertainty, but it was not eliminated. The atypical beach sediment at Sargent Beach and the loss of fines further complicate selection of an accurate conversion factor.

The macro-budget was used as a check to help verify sediment budget results over a large scale. It extended from Freeport to Brazos Beach to MSC, excluding Freeport and Matagorda Ship Channels. Only the residual for beach cells is in Table 2. The residual for the macro-budget equals the sum of the residual for all cells in the budget, excluding Freeport and Matagorda Ship Channels, verifying accurate accounting of sediment within SBAS.

Evaluation of the magnitude of the residual helps enable refinement of potential transport rates in the region. If the assumption that all volumetric change calculations are accurate is accepted, some potential reasons for these residuals can be inferred. Cell residuals show that more sediment accumulates between Brazos and Cedar Lakes, compared to the published values of longshore transport. Deposition of Brazos River sands and redistribution of the inlet shoals are the sources of this additional sediment (Fields et al. 1988); the additional transport capacity needed to deliver the sediment can be attributed to the increase in relative shoreline orientation as the river mouth has prograded.

The MCR to MSC: North cell has more sediment entering than shoreline change accounts for, and the MCR to MSC: South cell has more leaving. Therefore, it is likely that net transport is greater to the west here than estimated, which would account for the difference.

There is consistently more erosion between west of Cedar Lakes to west of Mitchell's Cut than is included in the preliminary budget. A total of 878,575 yd^3/yr is not yet accounted for in the budget for this reach (similar to results in Seelig and Sorenson 1973).

This updated conceptual sediment budget represents the average conditions for a recent period within the region. The main conclusions applied for project design are:

- Insufficient sand-sized sediment is available to maintain shoreline position, leading to overall shoreline retreat throughout the region.
- Sediment supplied by the Brazos River is deposited in a relatively small region surrounding the river mouth and transported to adjacent beaches by energetic waves and currents. Future sediment supply from the river is unknown.
- The sediment budget highlights poorly understood processes at Sargent Beach responsible for the extreme erosion in the area. As stated, the cause likely is a combination of cross-shore transport, fine-grained sediments, and trapping at inlets (Mitchell's Cut and several ephemeral inlets formed during storms).
- The sediment budget west of Mitchell's Cut is reasonably well balanced. Uncertainty in volume change and transport explain the observed residual.

2.2   Mitchell's Cut channel equilibrium area analysis

The Channel Equilibrium Analysis (CEA) tool (http://cirp.usace.army.mil/products/ index.htm) was used to evaluate the stability of Mitchell's Cut. If the channel is at equilibrium between the oceanside and bayside forcing, it will tend to retain the same area in response to perturbations caused by adjacent impacts such as a beach nourishment project. Campbell et al. (2005) noted that many inlets in Louisiana are sand starved, and this needs to be considered in coastal restoration plans. Answering this question can help facilitate engineering strategies for Sargent Beach. Performing the CEA calculation for Mitchell's Cut is the means to address this issue.

The actual bay area is difficult to measure and depends on the particular wind patterns. The CEA program requires the cross-sectional area and depth as input. Therefore, computed discharge results from the validated CMS model were used. A channel width of 330 ft (100 m) and length of 3,775 ft (1,150 m) were used. The CEA program gave a value of 41,792,152 ft² (3,882,618 m²) for the equilibrium area, which is about half the measured value. This implies the inlet is sand starved and could come close to its equilibrium area if a sudden influx of sediment were to occur at Mitchell's Cut.

However, the discharge from the inlet is not constant (Figure 2). Flooding and storms can greatly increase the discharge over the values used here, enabling the stability of a wider channel. The values presented here are consistent with the value of 7,060 ft³/sec (200 m³/sec) given by Kraus and Militello (1999) as a typical peak daily discharge at Mitchell's Cut. Furthermore, Kraus and Militello (1999) speculated, "… that infrequent heavy precipitation and subsequent strong discharges from Caney Creek might be a significant factor in maintaining the stability of Mitchell's Cut."

Figure 2. Mitchell's Cut flow direction and computed discharge.
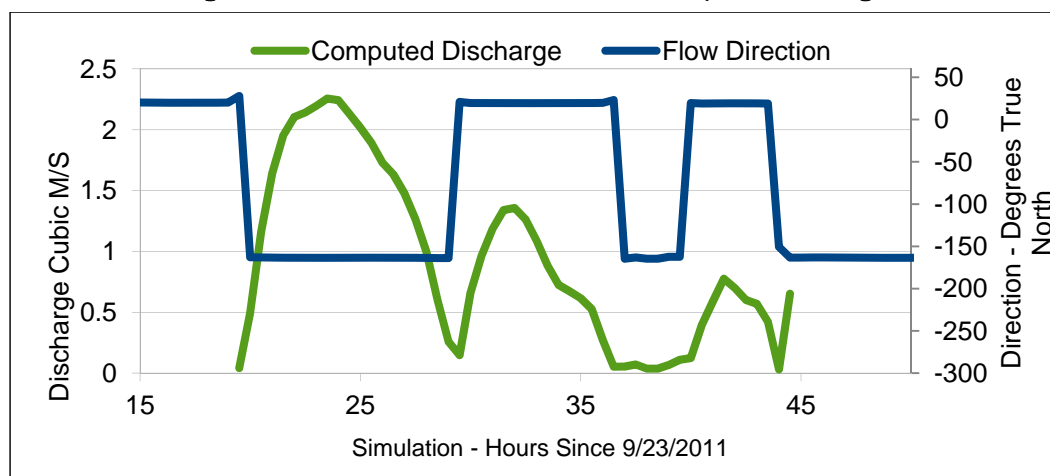
The results of this method suggest Mitchell's Cut would remain open if a beach fill were placed near the inlet, but temporarily it could narrow somewhat, depending on the incident storms.

2.3   Brazos River influence on shoreline change

The Brazos River is the single largest supplier of sediment to Texas beaches and is the dominant source of sediment to Sargent Beach not originating from adjacent beaches. The river's delta has been in a state of

evolution since it was diverted in 1929 (Odem 1953; DeWitt 1985; Fields et al. 1988; Hamilton 1995; Kraus and Lin 2002).

Kraus and Lin (2002) suggested the new delta reached a state of dynamic equilibrium in the 1960s, allowing the maximum possible natural bypassing. Sediment tends to be delivered by the river episodically during floods, as well as through longshore transport from adjacent beaches; net transport is to the southwest, with sediment historically supplied by the now-deflated ebb shoal at Freeport. Then, sediment is bypassed out of and around the delta during periods of larger waves.

The historical sediment budget indicated approximately 750,000 yd^3/yr were trapped between the Brazos and San Bernard rivers in accreting shorelines. More recent data between 1991 and 2011 indicate approximately 54,000 yd^3/yr has been trapped in accreting shorelines in this region. Additionally, 277,000 yd^3/yr were trapped in accreting shorelines between the San Bernard River and Cedar Lakes Pass during the recent period, compared to a net erosion of 206,000 yd^3/yr during the historical period after the river's diversion. These data appear to indicate shoreline change is stabilizing near the Brazos River as more sediment is bypassed, and that the excess sediment then is being trapped in adjacent beaches that had been sand starved.

The observations just mentioned could lead to the conclusion that the phenomenon will continue to progress to the west as beaches reach a new post-river diversion equilibrium, ultimately leading to reduced erosion rates at Sargent Beach. However, Seelig and Sorenson (1973) estimated two-thirds of the original sediment supply of the Brazos River to the Gulf of Mexico was reduced by reservoir construction in the 1950s and 1960s, decreasing the sediment available to adjacent beaches. This drastic change in sediment supply makes it impossible to relate the equilibrium conditions of the previous discharge location to those expected after diversion.

In conclusion, results presented by many others suggest the Brazos River delta has reached its post-diversion equilibrium. Limited analysis conducted during this study suggests sediment bypassing the delta is helping stabilize shorelines farther downdrift, but has not yet measurably influenced erosion rates at Sargent Beach. Because of the long period during which beaches and deltas evolve and the myriad anthropogenic and natural factors, further analyses and data collection, beyond the scope of this study,

would be required to state conclusively that the Brazos River diversion has influenced erosion rates at Sargent Beach. This study will apply recent (1991 to 2011) data, which appear to illustrate the river delta is in a state of dynamic equilibrium, to develop numerical and analytical models on which design decisions will be based.

3   Numerical Modeling Calibration and Setup

Numerical models were used to validate and predict shoreline change and morphology. GenCade was applied to model longshore transport and shoreline change. The Coastal Modeling System (CMS) (http://cirp.usace.army.mil/products/index.html) was used to evaluate with-project changes to waves and currents at the entrance to the inlet and to compute detailed wave heights and wave and current patterns around the proposed breakwaters. This chapter describes the setup and calibration of numerical models applied during the design process.

3.1   Shoreline change model

This section summarizes development of GenCade, a shoreline change model. It will be applied to facilitate alternative selection and design. Model setup and analyses are discussed in Appendix A.

3.1.1   Model description and limitations

GenCade is a one-line model of shoreline change with the added capability of inlet volume evolution (Frey et al. 2012). Coupling the inlet model with the one-line model makes it possible to apply GenCade over larger spatial scales without the need for multiple grids. It also adds functionality to relate inlet volume deficits to shoreline change. GenCade can simulate shoreline response to beach nourishment; inlet dredging; construction of groins, jetties, and breakwaters; and changes in the wave climate. GenCade, Version 1, Release 1, was used for this analysis and the MCR analysis. Application of breakwaters, groins, and beach fills are in development in this version of the code. GenCade, Version 1, Release 2, which improved the transport routine when breakwaters are present, was used for the alternatives at Sargent Beach.

GenCade is constrained by the standard assumptions upon which one-line models are based (Frey et al. 2012):

- The beach profile remains constant.
- The shoreward and seaward depth limits of the profile, the berm height and depth of closure, respectively, are constant.

- Sand is transported alongshore by breaking waves and longshore currents.
- The detailed structure of the nearshore circulation is ignored.
- A long-term trend in shoreline evolution is present.

The complex processes at Sargent Beach related to the presence of cohesive sediments clearly stretch the aforementioned assumptions. However, no better model exists to evaluate the long-term function of proposed alternatives. Therefore, it is important to consider the model results qualitatively (in a relative manner) and to apply engineering judgment in their application to design.

3.1.1.1   Units, coordinate system, datum

Standard International (SI) units are applied in all model runs. Units are converted to the US Customary System in this report. The horizontal coordinate system is Universal Transverse Mercator (UTM), Zone 14. The horizontal datum is NAD83. The vertical datum is NAVD88.

3.1.1.2   Direction convention

The GenCade grid is aligned so the water is on the left side of the grid when facing the positive direction. When facing the water, transport is negative to the left and positive to the right. Waves can be imported in any sign convention; the model automatically converts to grid normal.

3.1.2 Model domain

The GenCade model domain (Figure 3) extends from SBR to MSC. It contains 655 grid cells of variable size from 130 to 490 ft (40 to 150 m) with smaller cells near structures and inlets. The total length of the grid is approximately 54.75 miles (94.5 km). The GenCade model origin is less than 1 mile southwest of the SBR mouth. Symbols on the grid baseline indicate wave gauges are perpendicularly offshore (Figure 3).

3.1.3 Calibration and validation summary

Nearly all the input and model parameters specified in the Phase 1 study were applied in Phase 2. The main exceptions were K1 and K2 (sand transport rate coefficients) and ISMOOTH (the number of cells in the offshore contour smoothing window). Appendix A provides more details. One parameter that should be mentioned is the specification of D_{50}, the

Figure 3. GenCade model domain.
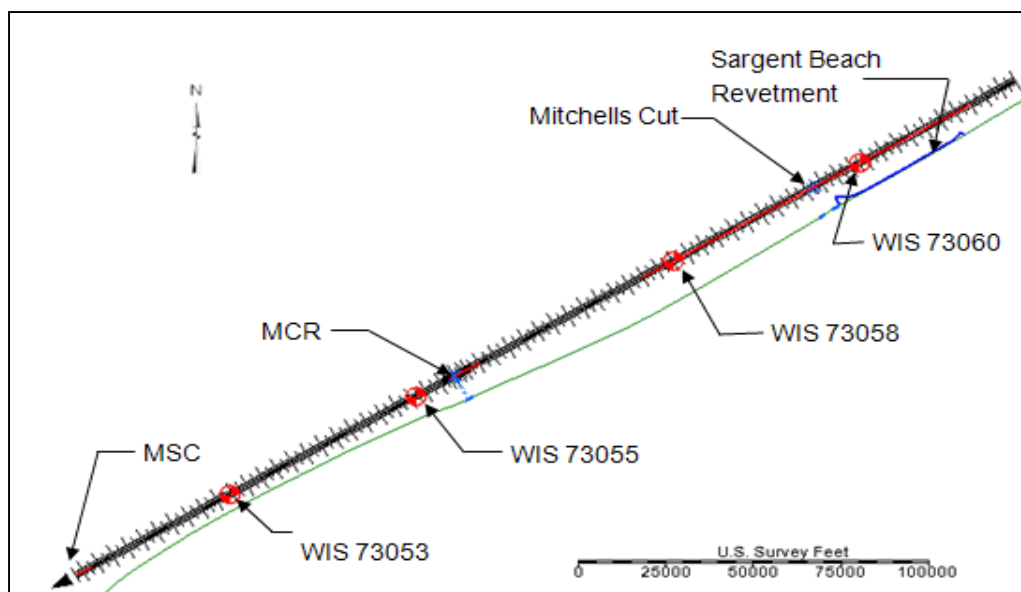
effective grain size in millimeters. For the GenCade calibration, validation, and subsequent alternatives, the effective grain size used in the model was 0.2 mm. This was based on previous sediment sampling in the area. A newer sediment sampling found an effective grain size of 0.14 mm. Although the GenCade simulations use the larger effective grain size of 0.2 mm, sensitivity tests for 5- and 16-yr -long simulations were conducted during calibration and found the effective grain size does not have a significant impact on the calculated shoreline change or transport rates.

Appendix A details GenCade model calibration and validation. Several GenCade model setups were simulated. First, the 1995 to 2000 simulation described in the Phase 1 report was recalibrated. Then, the 1991 to 1995 and 1991 to 2000 periods were simulated.

Figure 4 shows model results compared to shoreline change values published for the 1991 to 2000 period. The poorer fit near MCR (around 185+000 ft) is likely due to the orientation of the jetties. A jetty in GenCade is assigned to a specific cell. Therefore, each jetty must be perpendicular to the grid which is approximately parallel to the shoreline. There is about a 45° difference between the jetties in GenCade and the actual jetty orientation which affects shoreline change in the vicinity of MCR. Figures 5 and 6 compare the calculated net and gross transport rates to published ones. The model was calibrated from 1995 to 2000, and the 1991 to 2000 period gave the best results during validation. The 1991 to 1995 period produced the poorest results, which can be attributed to differing shoreline trends in some areas.

Figure 4. Shoreline change: model results vs. published values for the 1991 to 2000 case.
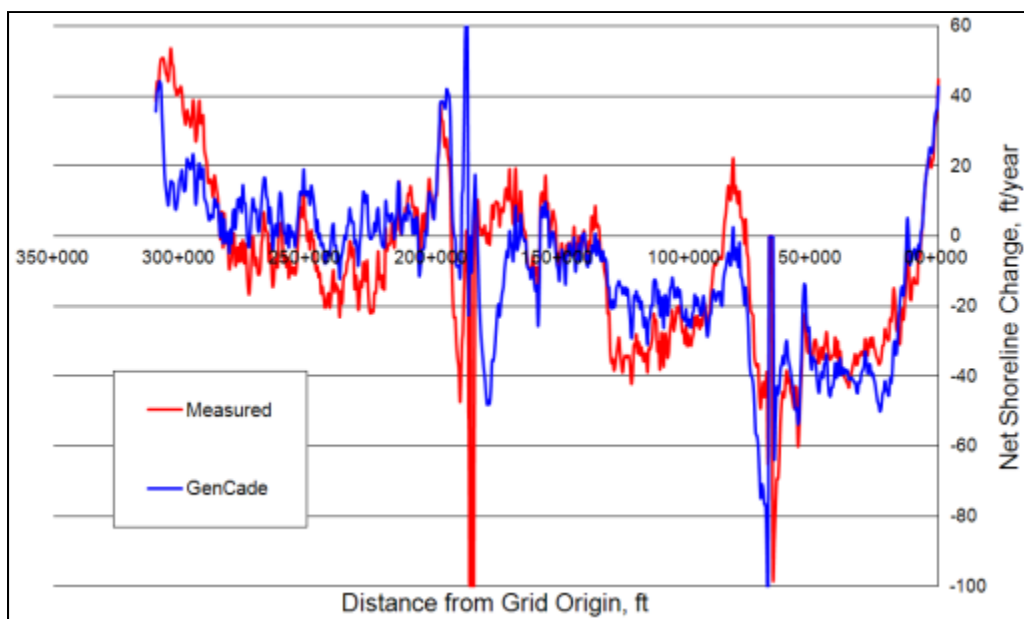
Figure 5. Net transport: model results vs. published values for the 1991 to 2000 case.
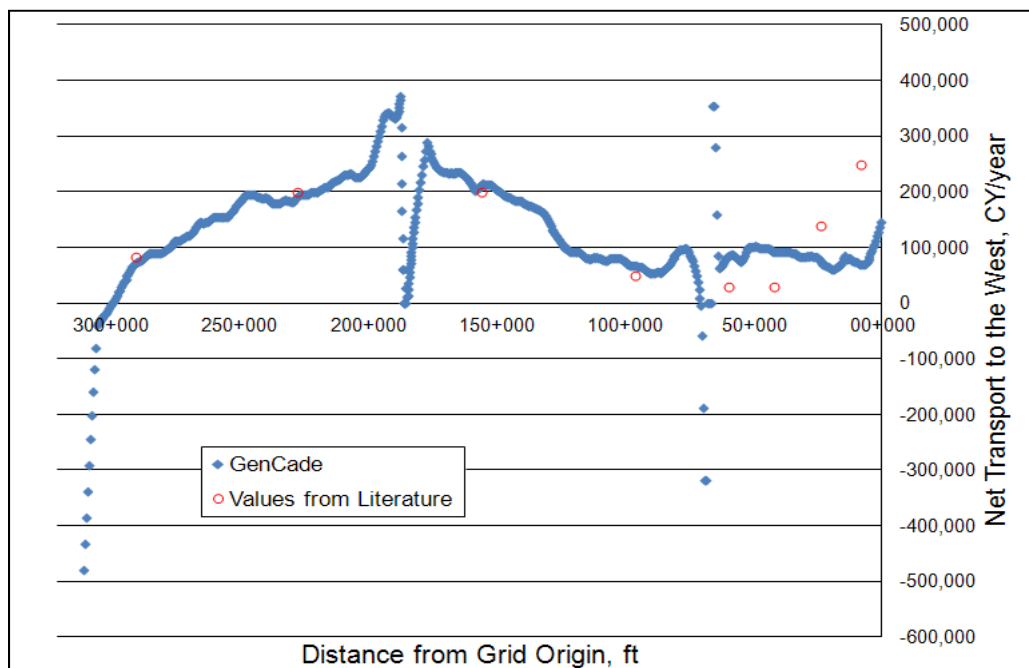
Figure 6. Gross transport: model results vs. published values for the 1991 to 2000 case.
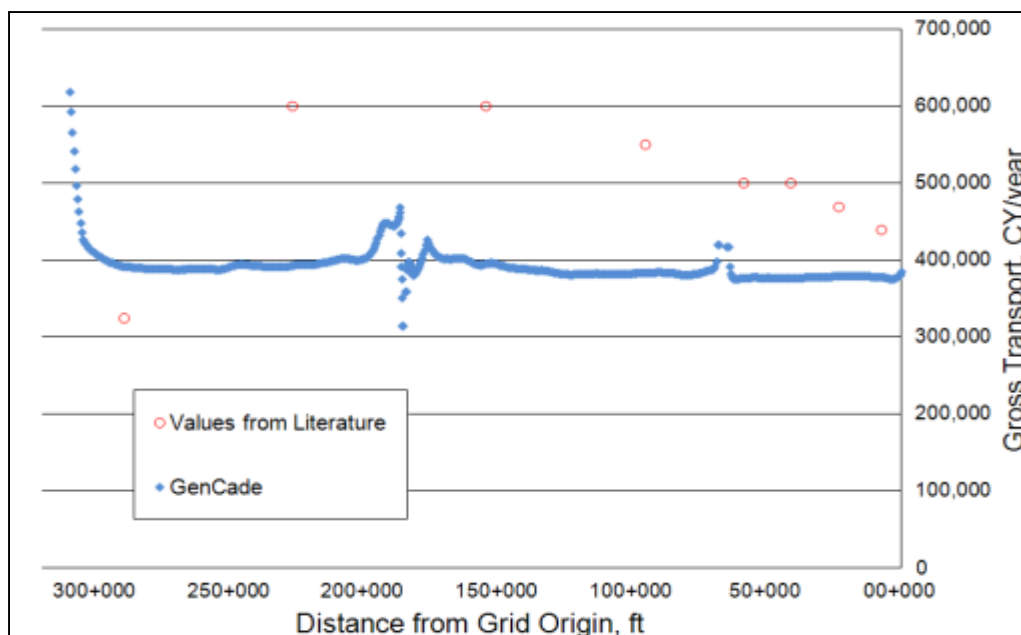
Tables 3 through 5 describe the calibration and validation statistics. Table 3 shows the statistics for the 1995 to 2000 calibration. Tables 4 and 5, respectively, show the validation statistics for the 1991 to 2000 and 1991 to 1995 cases. The Root-Mean-Squared Error (RMSE) and the Brier Skill Score provide goodness-of-fit statistics and scores for the GenCade results. The RMS Error is the difference between the measured and modeled shoreline change. The Brier Skill Score (BSS) reflects the level of agreement between the measured and calculated values. A BSS of 1 means the calculated and measured values are in perfect agreement; a value between 0.8 and 1 is excellent; and a value less than 0.3 is poor (USACE 2012).

Table 3. Shoreline change modeling statistics (1995 to 2000).

Cell	Average Shoreline Change (ft/yr)		RMS Error (ft/yr)	Brier Skill Score
	Measured	Modeled		
SBR to Cedar Lakes	10.4	7.4	7.4	0.87
West of Cedar Lakes	-17.2	-22.3	6.5	0.86
Sargent: East of FM 457	-23.9	-22.3	4.5	0.97
Sargent: West of FM 457	-26.1	-23.1	5	0.96
West of Mitchell's Cut	-18.6	-14.2	13.7	0.51
East of MCR	-5.7	-6.2	10.4	-0.10
MCR to MSC: North	-7.2	4.2	12.5	-0.57
MCR to MSC: South	6	6.7	6.9	0.54

Table 4. Shoreline change modeling statistics (1991 to 2000).

Cell	Average Shoreline Change (ft/yr)		RMS Error (ft/yr)	Brier Skill Score
	Measured	Modeled		
SBR to Cedar Lakes	1.8	6.7	6.3	0.90
West of Cedar Lakes	-29.6	-39.7	11.3	0.86
Sargent: East of FM 457	-34.8	-39.3	6	0.97
Sargent: West of FM 457	-49.1	-36.8	12.1	0.94
West of Mitchell's Cut	-21.8	-23.9	13.7	0.74
East of MCR	-1.8	-9.5	17.2	-0.84
MCR to MSC: North	-4.3	8.1	19.1	-0.76
MCR to MSC: South	19.6	12.2	17.5	0.64

Table 5. Shoreline change modeling statistics (1991 to 1995).

Cell	Average Shoreline Change (ft/yr)		RMS Error (ft/yr)	Brier Skill Score
	Measured	Modeled		
SBR to Cedar Lakes	-8.7	0.2	10.5	0.09
West of Cedar Lakes	-12.4	-18.1	7.2	0.69
Sargent: East of FM 457	-10.9	-17.9	8.7	0.47
Sargent: West of FM 457	-23	-19.1	6.1	0.93
West of Mitchell's Cut	-3.2	-11.4	16.4	0.16
East of MCR	3.6	-4.1	13	-0.60
MCR to MSC: North	2.9	4.4	16.7	-0.89
MCR to MSC: South	13.6	5.4	15.2	0.41

3.2   Wave, current, and sediment transport model

This section summarizes development of CMS, a numerical model of waves, currents, and sediment transport. The model will be applied to facilitate alternative selection and design in the following chapters. Model setup and analyses are discussed in Appendix B.

3.2.1 CMS domain and setup

Figure 7 shows the extent of the model grid used for CMS. The larger grid was used to compute water levels and currents. Water levels from the larger grid solution then are extracted and used to force boundaries of the nested grid. To better compute sediment transport in the areas of interest, the nested grid uses smaller cells (higher resolution) to more accurately represent structures, bathymetry, and coastal processes. Waves are computed with the nested grid and with corresponding high resolution (Appendix B).

Figure 7. CMS large grid and nested grid extent.
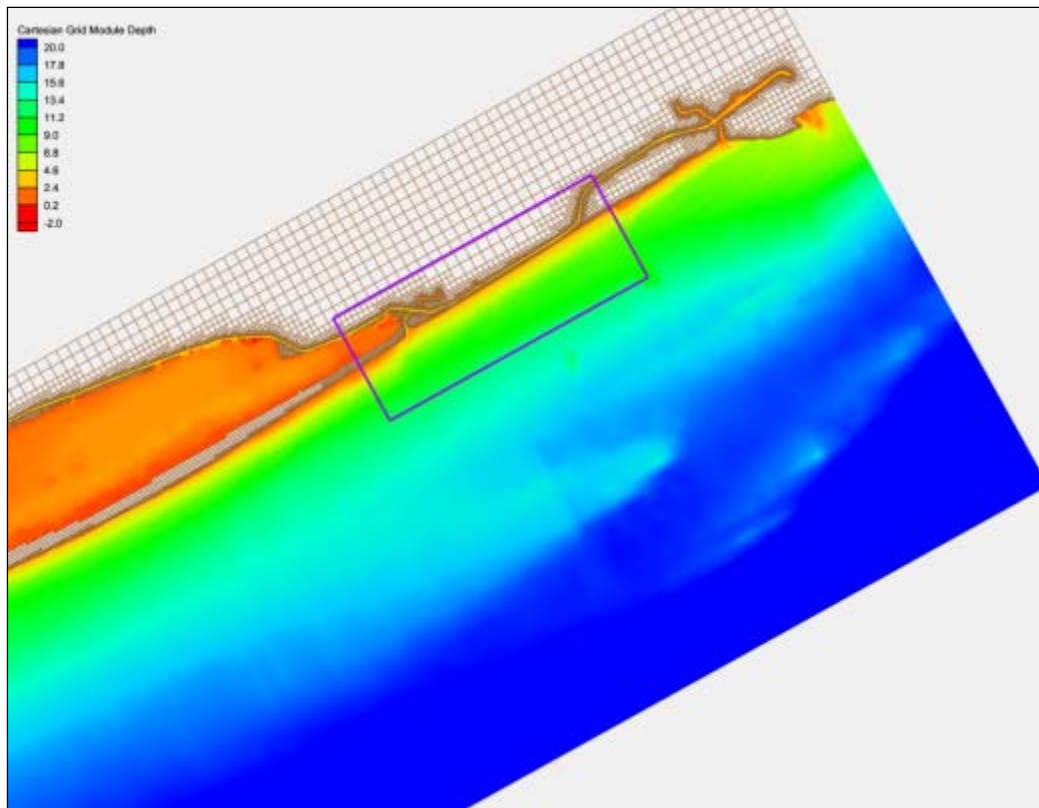
3.2.2 Tides and currents

The larger model was validated against water levels measured at the FM 457 Bridge over the GIWW. The model was calibrated with water level measurements from September 23 to October 1, 2011. The model was validated from October 1 to 30, 2011 (Figure 8). Goodness-of-fit (USACE 2012) validation statistics are in Table 6. A major driver of water levels in Texas bays is the wind (Kraus et al. 2006). Insufficient data exist to describe the spatially varying wind field, so single point measurements were used to specify a uniform wind field over the model grid. This lack of wind data appears to be the major cause of differences between computed and measured values. Supporting information is in Appendix B.

Current data were not available to validate the model. However, water level validation is more important because currents accompany changes in water levels. Section 2.2 refers to CMS computed discharge values, which are dependent on the current, that are consistent with the values in Kraus and Militello (1999). Computed longshore currents appear to be reasonable, as discussed in Section 5.3.2.4.

Figure 8. Measured data compared to CMS model results.
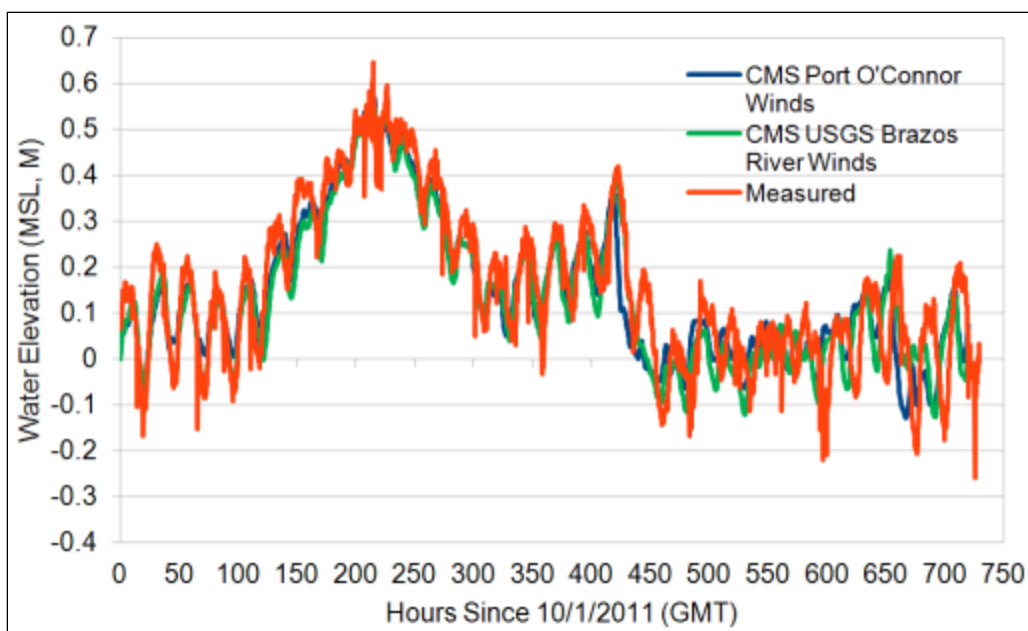
Table 6. CMS water level validation statistics.

CMS Run	Bias (m)	R^2	RMSE (m)	MAE (m)
Port O'Connor Winds	0.0159	0.8879	0.0796	0.2478
USGS Brazos River Winds	0.0241	0.9029	0.0767	0.2451

RMSE = Root-Mean-Squared Error; R^2 = squared correlation coefficient; Bias = mean difference between measured and calculated; MAE = mean average error.

3.2.3 Waves

Design waves were checked using the CMS-Wave model in standalone mode. A Joint North Sea Wave Project (JONSWAP) spectrum of the maximum wave condition, defined in Thomas and Dunkin (2012), was defined using the Surface-water Modeling System (SMS) wave editor. The water levels were varied up to the 50-yr design height (Thomas and Dunkin 2012) and were defined in the spectral input file. Different versions of the wave grid were produced for both Sargent Beach and MCR, each having a grid resolution between 10 and 20 m in the vicinity of the grid where structural placement is considered. Figure 9 shows an example run with wave vectors and linear contours of the wave height. Waves measured during the validation period and in June 2008 were used to represent typical conditions for sediment transport modeling.

3.2.4 Sediment transport and morphology

The combined flow, waves, and sediment transport of CMS is especially useful in the vicinity of Mitchell's Cut and for exploring detailed processes around the breakwater structures. CMS does not include swash zone (shoreline) processes but, due to the gentle 100H:1V to 200H:1V beach slopes, the largest contribution to sediment transport is the relatively wide surfzone, not the shoreline. Short-term CMS results are used to search for trends under typical environmental forcings that violate GenCade (which gives long-term results) model assumptions.

Figure 9. Example CMS-Wave wave height and direction results with 50-yr design waves and varying water level.
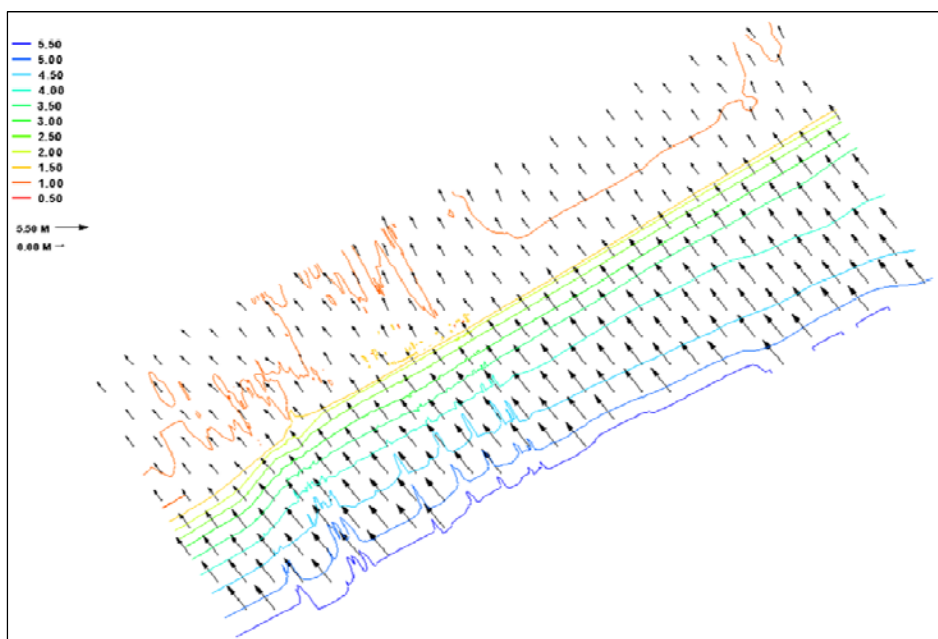
Comparative survey data of Mitchell's Cut and East Matagorda Bay are not available to calibrate CMS with respect to sediment transport. Sediment in the inlet tends to selectively sort finer sediment fractions and act more resistant to erosion than the typical D_{50} grain size would indicate. CMS has a mechanism to specify the transport grain size while varying the sediment D_{50} according to location (Sanchez 2012). In Figure 10, a D_{50} of 2 mm was specified within Mitchell's Cut, with a transport grain size of 0.14 mm. The erosion seen within Mitchell's Cut is unrealistic, but the general pattern of sediment transport is as expected (Chapter 5). The structures within this domain were classified as non-erodible within CMS. However, scour effects are evident on the adjacent bottom, showing the necessity of scour protection in the design.

Figure 10. CMS example: October 1 to 30, 2011, forcing.
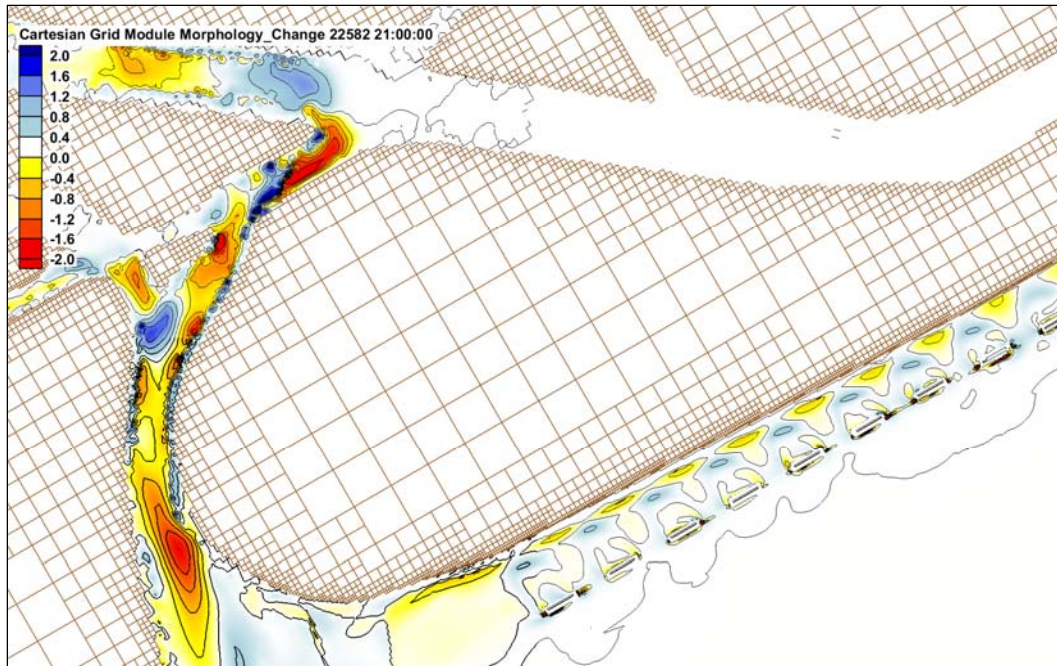
4   Matagorda Peninsula

This chapter describes the process to select a structural alternative near MCR. Several preliminary alternatives are described, and GenCade results are discussed. A preliminary groin system design with beach fill for the recommended alternative is shown, and a preliminary opinion of construction cost is given.

4.1   Previous work and study goals

Kraus et al. (2008) documented the history of MCR and described the design of a new jetty, constructed in 2010. The objective of this part of the study was to reduce erosion and increase beach width over a defined target area at Matagorda Peninsula. The Phase 1 study model results indicated construction of a single groin likely would meet this goal, but that success would be highly dependent on bypassing operations at MCR. Because federal funding for dredging at shallow draft navigation channels is limited and uncertain, alternatives were suggested with and without an installed bypassing system. After presentation of the Phase 1 results to the Port of Bay City Authority, the final project goals were specified:

- Establish a groin field to stabilize the beach from the north access road to 3 Mile Cut.
- Increase the dry beach width by 200 ft over this area.
- Ensure no impact to shoreline change rates at 3 Mile Cut, an ephemeral inlet with environmental benefits.
- Exclude bypassing plants at MCR. Natural bypassing and dredging of MCR and placement near the west beach might occur (various levels of federally funded bypassing are considered in the shoreline modeling, although no additional non-federal bypassing was recommended or tested).

4.2   Definition of alternatives

As noted in the Phase 1 study, it is challenging to reach a solution that meets the desired objective of a 200-ft-wide beach. To attempt to do so, 10 alternatives were modeled (Table 7) in addition to the existing condition. The initial groin configurations for Alternatives 1 through 5 resulted in accretion near 3 Mile Cut, which was undesirable because it could change

the way the ephemeral inlet has functioned. Therefore, each alternative was rerun with the groin field shifted 1,640 ft (500 m) to the southwest. Alternative 5-Shifted was the best performing alternative. See Table A6 for a summary of the results. Discussion, groin configuration, and shoreline change results for Alternatives 1 through 4 and Alternatives 1 through 4-Shifted are in Appendix A.

Table 7. MCR alternatives.

Alternative	Description
0	No action, no groins constructed
1	Seven groins, 400 ft long, 800 ft apart (last three groins shorter and closer together); first groin approximately 2,500 ft southwest of 3 Mile Cut
1-Shifted	Same as Alternative 1, groins shifted 1,640 ft to southwest
2	Five groins, 400 ft long, 1,200 ft apart (last two shorter and closer together); first groin 2,500 ft southwest of 3 Mile Cut
2-Shifted	Same as Alternative 2, groins shifted 1,640 ft to southwest
3	Five groins, 600 ft long, 1,200 ft apart (last two are shorter and closer together); first groin 2,500 ft southwest of 3 Mile Cut
3-Shifted	Same as Alternative 3, groins shifted 1,640 ft to southwest
4	Five groins, 600 ft long, 1,800 ft apart (last two shorter and closer together); first groin 2,500 ft southwest of 3 Mile Cut
4-Shifted	Same as Alternative 4, groins shifted 1,640 ft to southwest
5	Three groins, 800 ft long, 1,600 ft apart
5-Shifted*	Same as Alternative 5, groins shifted 1,640 ft to southwest

*This is the recommended configuration, shown in detail in the next section.

4.3   Numerical model results

This section describes the GenCade setup and results of the recommended groin system near MCR. Analysis of unselected alternatives is in Appendix A. Initially, the setup to calibrate and validate the GenCade model was used; however, more detail was necessary in the region near the groins. Therefore, higher resolution grid spacing was applied. The GenCade grid also was shortened to minimize the time to run each simulation. The model domain was 28 miles long, extending about 3.3 miles southwest of Mitchell's Cut to about 8.7 miles southwest of MCR (Figure 11).

Since the grid is shorter than the grid used for model calibration and validation, only data from the Wave Information Studies (WIS) (Tracy 2004) hindcast stations 73055 and 73058 were applied. Applied also were the

Figure 11. GenCade MCR model domain.
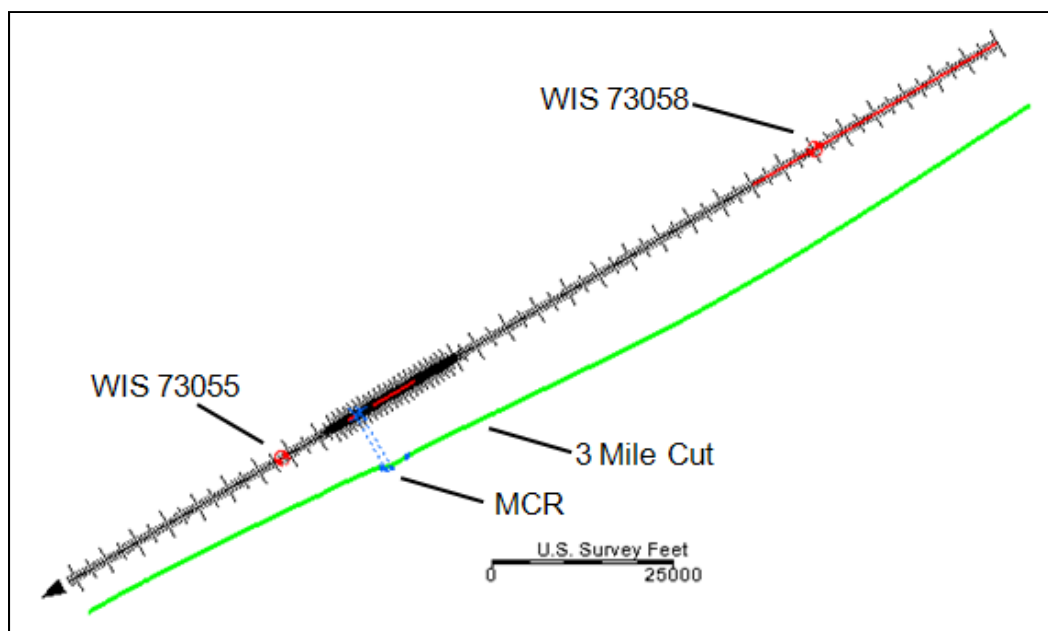
same calibration and validation parameters as in the initial grid. The grid used for the preliminary analysis of MCR in Phase I was longer. The alternatives only affected shorelines nearby and had no effects on shoreline change or longshore transport outside of that area. In order to reduce the amount of time necessary to simulate the alternatives, the grid was reduced to a shorter length where parts of the grid that were removed experienced little to no change in shoreline position or transport rates. The 1995 to 2000 data were applied first. In these cases, the 1995 shoreline was applied as the initial shoreline. These short cases were used to refine the grid spacing in the area of interest and determine the effects of groins in the short term. A 16-yr period also was simulated. Since the WIS hindcast spans from 1980 to 1999 only, waves were repeated to represent the years from 2000 to 2011. The 16-yr cases were simulated to model the effects of the groin alternatives over a longer period.

Once the grid spacing was refined, the 5- and 16-yr-long simulations were conducted, with the 2011 shoreline position applied as the initial shoreline position. Due to the small grid spacing in the area of interest, the shoreline was smoothed. Without smoothing, the calculated shoreline becomes slightly jagged and the results are more difficult to interpret in the plots. Required smoothing and grid modifications induce greater uncertainty than reported in Chapter 3; however, this approach was acceptable because the results were applied in a qualitative and relative manner.

The future rate of natural, or possibly mechanical, bypassing is unknown, so a range of values were used to represent natural bypassing or federally funded dredging or bypassing. Because it is unlikely the projected required sediment bypassing of 200,000 yd^3/yr (Kraus et al. 2008) will occur naturally, the bypassing rate was decreased to 115,000 yd^3/yr for many of the simulations and assumed to be the natural bypassing rate or mechanical placement of dredged sediment from MCR near the southwest beach. Sediment bypassing of 0, 115,000, and 200,000 yd^3/yr were simulated for the recommended configuration. To increase the clarity of the figures in this section, only Alternative 5-Shifted with 0 and 200,000 yd^3/yr of bypassing are shown. Alternatives 1 through 4 are described in Appendix A; the bypassing rate specified for those runs was 115,000 yd^3/yr. The sediment bypassing operation east of MSC was outside of the boundaries of this shortened grid, so it is not included. The next sections describe the numerical modeling results for Alternative 0 and the final groin configuration, Alternative 5-Shifted.

4.3.1 Non-structural alternatives

The baseline action is to take no action. Figure 12 shows the setup in the region of interest without any action. The green line represents the initial shoreline. The calculated shoreline is difficult to see at this scale, so it is not included in Figure 12; results after 5 yr indicated net erosion equal to -10 ft/yr.

Figure 12. GenCade setup between MCR and 3 Mile Cut for Alternative 0.

In addition to the no-action case, three variations without structures were simulated for comparison to the different variations of Alternative 5-Shifted. The four cases described in this section are:

- No action: no beach fill, no mechanical bypassing;
- 200-ft-wide beach fill, no mechanical bypassing;
- No beach fill, 200,000 yd^3/yr mechanical bypassing through dredging and downdrift placement;
- 200-ft-wide beach fill, 200,000 yd^3/yr mechanical bypassing through dredging and downdrift placement.

It was assumed mechanical bypassing would be the result of a federal navigation channel dredging project. Also, inlet processes are not well known because of construction of the new east jetty. It is likely that more natural bypassing will occur now that the new jetty is in place. In the following results, mechanical bypassing should also be understood to include increased natural bypassing as a result of the new jetty.

The beach fill in GenCade only extends 3,280 ft along shore. This is the length between the first and last groin in Alternative 5-Shifted. Some sensitivity testing was conducted in GenCade, and it was determined that this setup provided the best combination of shoreline advance at the project site and minimal accretion at 3 Mile Cut and the MCR. Accretion at both locations is undesirable because accretion at 3 Mile Cut could impact historical performance of the ephemeral inlet and accretion at MCR could result in reduced recreational value or erosion on the adjacent beach. If the beach fill began north of the first groin, 3 Mile Cut accreted. A longer beach fill with an added width of 200 ft would cause additional shoreline advance at the east jetty of MCR, which eventually would increase dredging require-ments. A longer beach fill of the same volume provided slightly less protec-tion at the groins and increased the sand accumulated at the east jetty of MCR.

4.3.1.1   Shoreline change

Figure 13 plots the net shoreline change after 5 yr without any action. Positive values indicate accretion; negative values, erosion. Because the alternatives look at future scenarios, the initial shoreline in GenCade is specified as the 2011 shoreline.

If no structures are built, no mechanical bypassing occurs, and no beach fill is constructed, nearly the entire stretch from 3 Mile Cut to MCR experiences accretion (Figure 13). Large amounts of erosion occur directly southwest of MCR. Accretion northeast of MCR can be attributed to the lack of bypassing. If bypassing occurred close to the recommended rate, sand would not build up on the northeast side of MCR.

Figure 13. Simulated shoreline change between MCR and 3 Mile Cut for Alternative 0 after 5 yr.
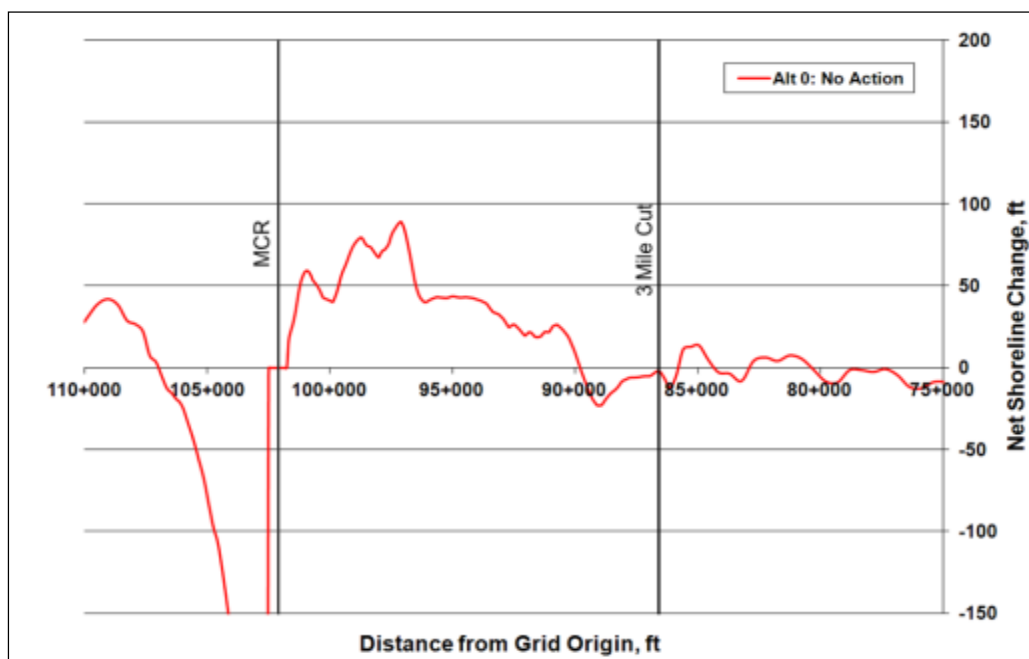
The figures in this chapter and in Appendix A do not show shoreline erosion southwest of MCR of more than about 150 ft. The reason this is not shown in the figures is because this study is more interested in the impacts between 3 Mile Cut and MCR. If the figures were expanded to show the extent of erosion to the southwest of MCR, it would have been extremely difficult to see the changes in shoreline position between 3 Mile Cut and MCR for the different alternatives. In addition, the structural alternatives have little effect on the shoreline to the southwest of MCR. Only in the cases with bypassing is shoreline erosion decreased to the southwest of MCR. When bypassing is not considered, the net shoreline change southwest of MCR is almost identical for the no action case and the alternatives, so the ranges of the figures were readjusted to highlight the differences in shoreline change between 3 Mile Cut and MCR for the cases.

Figure 14 compares each of the non-structural cases specified above. The effects of bypassing and a beach fill do not extend much further north than

3 Mile Cut. The greatest amount of accretion north of MCR occurs when a 200-ft-wide beach fill is constructed and no bypassing occurs. This scenario results in about 100 ft of shoreline advance for most of the region between 3 Mile Cut and MCR after 5 yr. Erosion occurs between 3 Mile Cut and MCR when 200,000 yd³/yr of bypassing occurs and no beach fill is added. The range of shoreline change over 5 yr is between 3 ft of erosion and 19 ft of accretion at 3 Mile Cut.

In addition to 5-yr-long simulations, GenCade was run for 16 yr. In the case with no beach fill and no mechanical bypassing, shoreline accretion steadily increases from 3 Mile Cut to MCR (Figure 15). Sixteen yr was chosen as the long-term simulation time period, because the model was run originally with the 1995 shoreline as the initial shoreline. For these 16-yr cases, 1995 to 1999 were represented with the actual waves. WIS Waves from 1991 to 1999 represented the period from 2000 to 2008. WIS Waves from 1991 to 1993 were repeated to complete the simulation for 2009 to 2011. The calculated shoreline was compared with the 2011 measured shoreline to determine how well the model reproduced the expected results. To simulate future scenarios, the 2011 shoreline was applied as the initial shoreline and the same 16-yr set of waves was used. Because different wave events could occur, GenCade results might not be representative of future conditions.

Figure 14. Simulated shoreline change between MCR and 3 Mile Cut for Alternative 0 cases after 5 yr.

Figure 15. Simulated shoreline change between MCR and 3 Mile Cut for Alternative 0 after 16 yr.
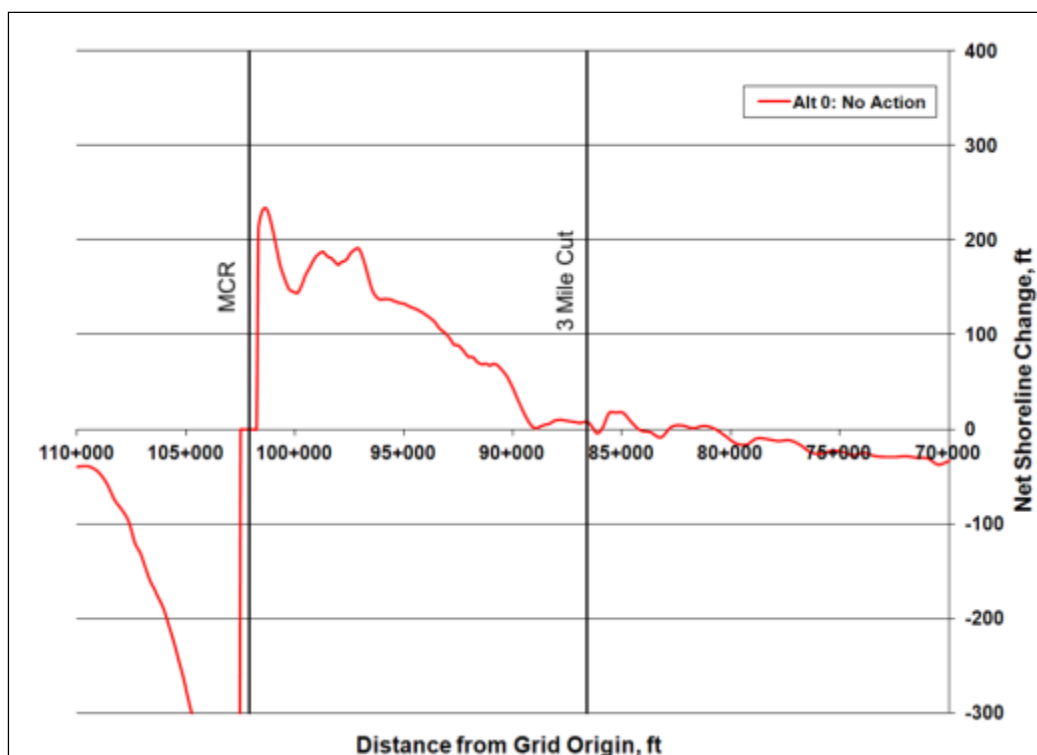
Figure 16 shows the four different Alternative 0 cases, to illustrate the effects bypassing and a beach fill had on the region during the 16-yr simulation period. Once again, the 200-ft wide beach fill and no bypassing case resulted in the greatest amount of accretion, and the no beach fill and 200,000 yd³/yr bypassing case resulted in the largest erosion. Even with 200,000 yd³/yr of mechanical bypassing, the shoreline just north of MCR advances almost 100 ft. At 3 Mile Cut, the range of shoreline change after 16 yr is from -17 ft to 35 ft.

Figure 16. Simulated shoreline change between MCR and 3 Mile Cut for Alternative 0 after 16 yr.
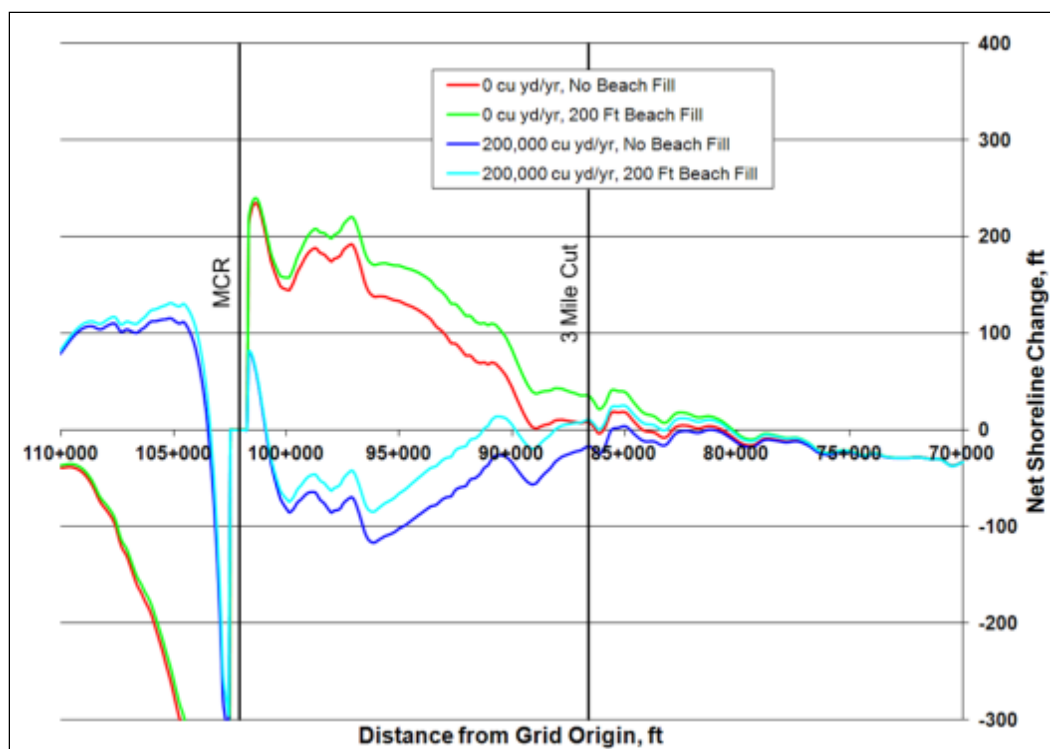
These results suggest no structures are needed to stabilize the shoreline in the target area. However, once MCR reaches a new state of equilibrium with the new jetty in place, assuming a navigable channel is maintained, it is likely the combination of natural and mechanical bypassing required to prevent flanking of the west jetty will be sufficient to induce net erosion in the target area (Kraus et al. 2008). The recommended structural solution is based on this assumption. Before construction, monitoring to verify this assumption must be conducted. The recommended alternative should be modified based on the results of data collected.

4.3.2 Alternative 5-Shifted: 800-ft groins, 1,600-ft spacing

This section details Alternative 5-Shifted, the preferred one. It was the last alternative configuration to be developed. The first four groin configurations did not meet the requirements of the project (200 ft of added dry beach width and no accretion at 3 Mile Cut), so they were not used. The groin layout for these alternatives was shifted about 1,640 ft southwest of the original alternatives, so the alternatives would not have as great an impact on 3 Mile Cut. These shifted alternatives did not result in the desired effects, so they also were removed from the discussion.

Alternative 5-Shifted consists of three 800-ft groins, 1,600 ft apart. The first groin in the original Alternative 5 configuration was about 2,500 ft southwest of 3 Mile Cut. This alternative impacted 3 Mile Cut negatively, so the groins were moved 1,640 ft southwest. This was the same procedure followed for the shifted Alternatives 1 through 4. Figure 17 shows the layout of the groins for Alternative 5-Shifted. Since Alternative 5-Shifted is the only one discussed in this document, it will be referred to as Alternative 5.

Figure 17. GenCade setup between MCR and 3 Mile Cut for Alternative 5 – Shifted.

Once the groin configuration was set, several different groin porosities were analyzed. A porosity of 0.3 was specified for the final design. It was chosen because it is typical of groin structures, precludes expensive sand tightening measures, and produces acceptable shoreline change.

Figure 18 compares Alternatives 5 and 0. Neither case features a beach fill or mechanical bypassing. Between 3 Mile Cut and the second groin, Alternative 5 results in shoreline advance. Downdrift of the third groin, Alternative 5 results in slight erosion, compared to the initial shoreline. West of the second groin, Alternative 0 results in more shoreline advance than Alternative 5.

Figure 18. Simulated shoreline change between MCR and 3 Mile Cut for
Alternatives 0 and 5 after 5 yr; groins located at vertical dashed lines.
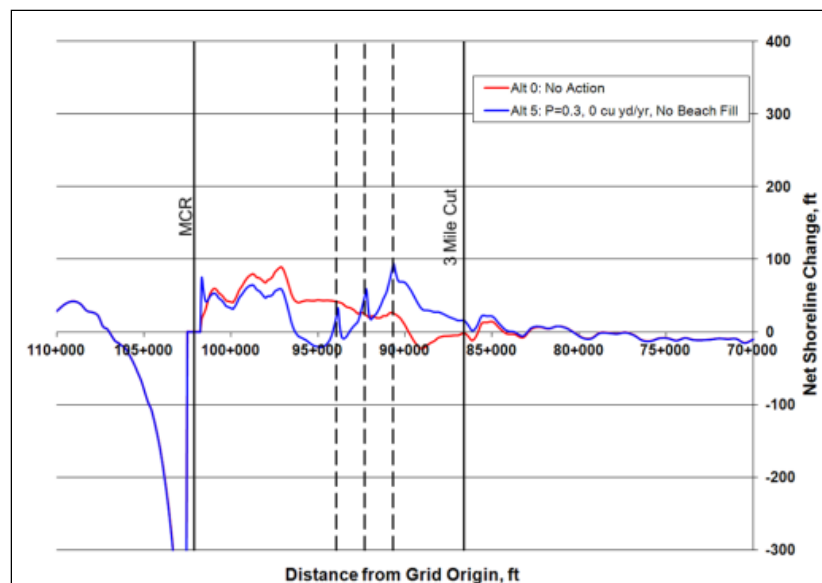
Figure 19 compares the same cases after 16 yr. For Alternative 5, almost
200 ft of beach advance occurs just east of the first groin. As expected, the
shoreline recedes to the west of the last groin, compared to the first groin;
however, the shoreline still advances from the initial shoreline. The reason
is the lack of bypassing around MCR. Alternative 0 results in more shoreline
advance, from the second groin to just east of MCR, than Alternative 5.

To see the range of calculated shorelines, four variations of Alternative 5
were simulated:

- No beach fill and 0 yd^3/yr of bypassing;
- No beach fill and 200,000 yd^3/yr of bypassing;
- 200-ft-wide beach fill and 0 yd^3/yr of bypassing;
- 200-ft-wide beach fill and 200,000 yd^3/yr of bypassing.

Figure 19. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 after 16 yr; groins located at vertical dashed lines.
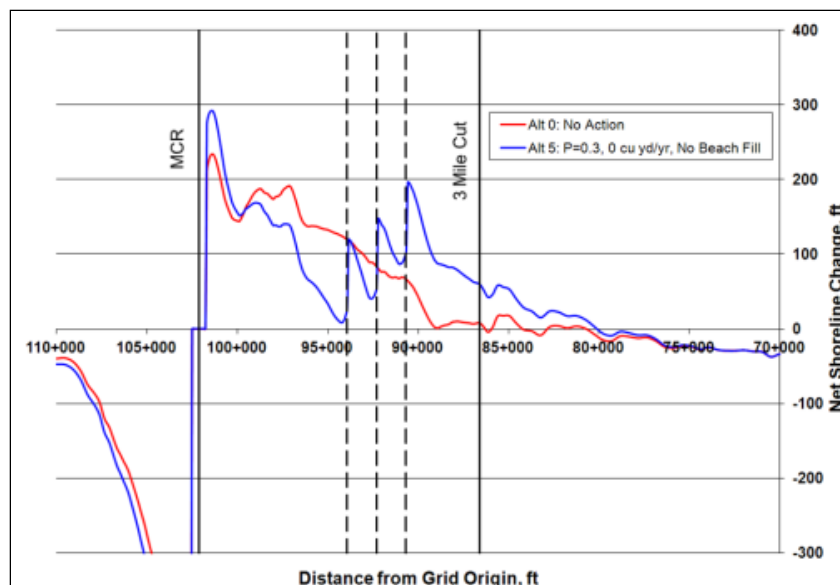
In Figure 20, the four scenarios after 5 yr are compared to Alternative 0 (no action, meaning no bypassing and no beach fill). Each alternative succeeds in widening the beach from 3 Mile Cut to the second groin. West of the third groin, only the case with the 200-ft-wide beach fill and 0 yd³/yr of bypassing maintains the shoreline comparable to the no-action alternative in this region, approximately halfway between 3 Mile Cut and MCR. The four variations of Alternative 5 result in a shoreline advance of 17 to 35 ft over 5 yr at 3 Mile Cut compared to Alternative 0. Before the first groin, the shoreline change ranges from 51 to 121 ft among the four cases. After the last groin, the shoreline change is -104 to 41 ft among the four cases.

A summary table in Appendix A features a column header labeled After Last Groin. The values for shoreline change in the table and in this discussion are calculated directly downdrift of the groin, so they do not identify the greatest amount of erosion. For example, the difference between Alternative 5 (no beach fill and 200,000 yd³/yr bypassing) and Alternative 0 (no action south of the third groin) is -62 ft. However, Alternative 5 (no beach fill and 200,000 yd³/yr of bypassing) erodes greater than 150 ft more than the final calculated shoreline for Alternative 0 (no action around 95+000 ft).

Figure 21 shows all the Alternative 5 cases after 16 yr. All, compared to Alternative 0, resulted in shoreline advance to the east of the first groin. Also, Alternative 0 resulted in greater shoreline advance than any of the Alternative 5 cases from the last groin to just east of MCR. At 3 Mile Cut, the

Figure 20. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all cases) after 5 yr.
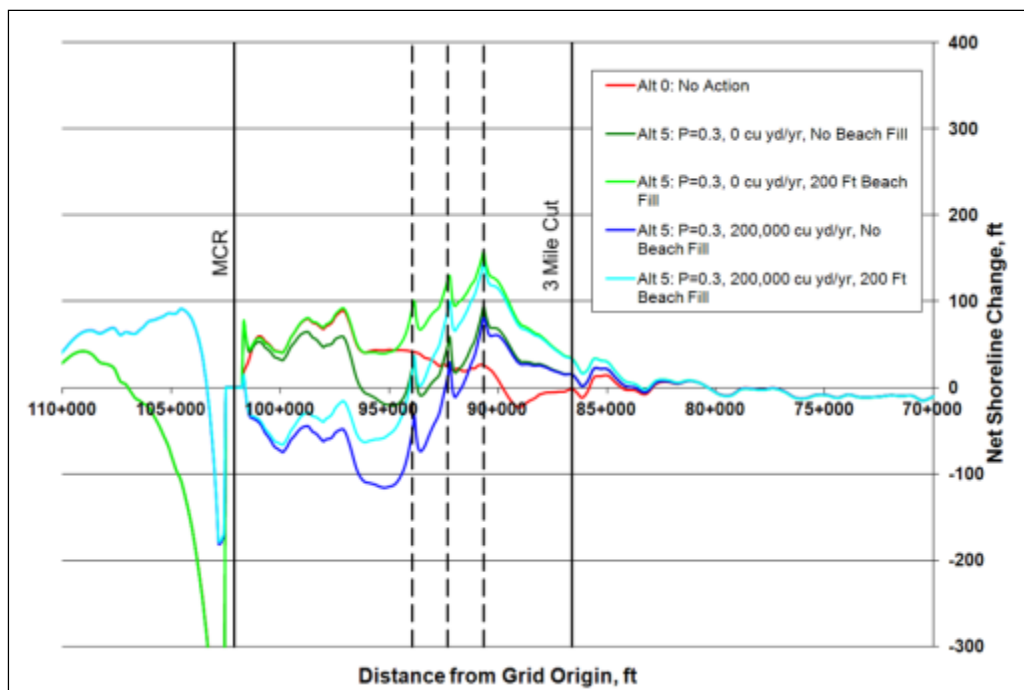
Figure 21. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all cases) after 16 yr.
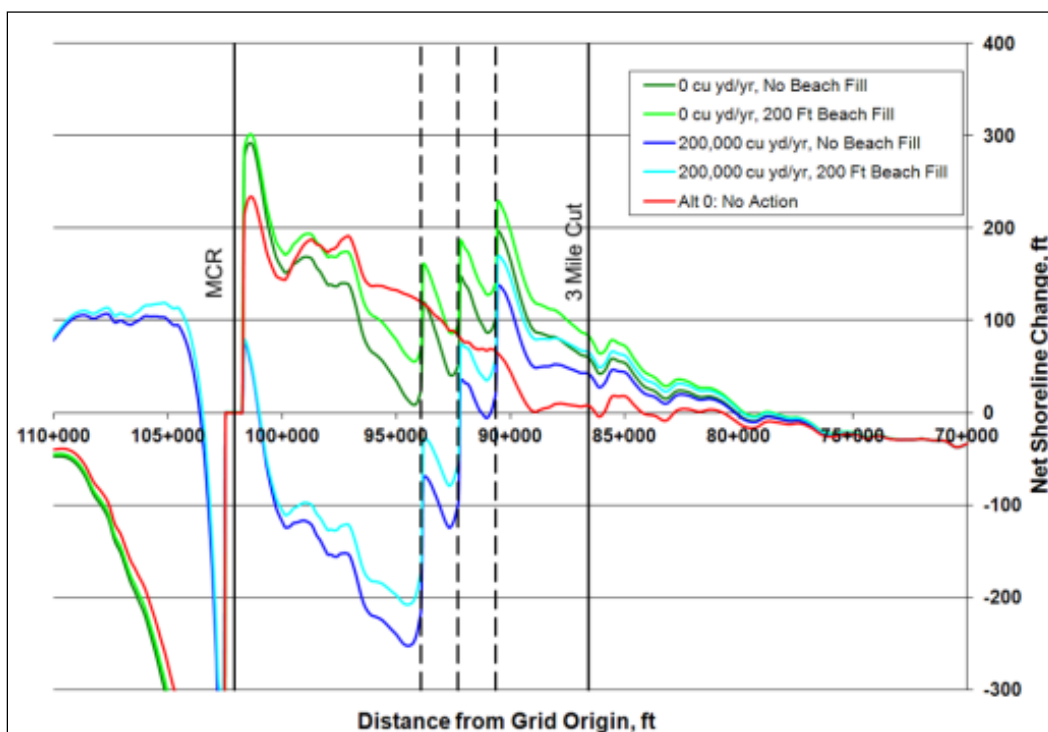
shoreline advanced between 34 and 75 ft. To the east of the first groin, shoreline advanced ranges from 72 and 164 ft from Alternative 0 – No Action. The greatest erosion downdrift of the last groin was nearly 250 ft from the initial shoreline when no beach fill was constructed, with 200,000 yd³/yr of bypassing. The 0 yd³/yr bypassing and 200 ft beach fill case resulted in about 64 ft of shoreline advance from the initial shoreline to west of the last groin. Keep in mind, different bypassing rates and different beach fill widths will adjust the final calculated shoreline.

Regardless of the amount of bypassing and the construction of a beach fill, the shoreline will advance updrift and erode downdrift of the groin field. None of the alternatives modeled resulted in the suggested 200 ft of shoreline advance across the range of the project, even when a small beach fill was added. Of all of those modeled, Alternative 5 provided shoreline advance most similar to the recommended project requirements. Based on the Alternative 5 results, mechanical bypassing is necessary to prevent shoreline advance north and erosion south of MCR. The beach fills modeled in GenCade were small in terms of volume and alongshore length, so increasing the sand volume and length of the fill was considered. However, over time, a larger beach fill most likely will affect 3 Mile Cut and MCR.

The modeled alternatives never resulted in consistent 200-ft shoreline advance. However, the groin configuration did result in consistent advance and a stable shoreline. Therefore, three beach fill options were provided in addition to the recommended groins. Figure 22 shows the calculated performance for each option after 16 yr. The wide range of results between bypassing and no bypassing illustrates the need for monitoring at MCR. Monitoring data should be analyzed to help determine which, if any, of the fill options should be selected.

4.4   Discussion of groin system feasibility

A series of groin configurations were analyzed to help widen the beach east of MCR, described in the previous section. The Port of Bay City Authority identified desired success criteria of up to 200 ft of additional dry beach from 3 Mile Cut to about 1 mile east of MCR, with no shoreline change at 3 Mile Cut to prevent changes to the ephemeral inlet there.

Sediment bypassing MCR is important to prevent flanking of the west jetty and to help manage accretion on the east side of the inlet. It also is critical to performance of the groin field. The new jetty design is based on

Figure 22. Net shoreline change after 16 yr with the recommended groin system.
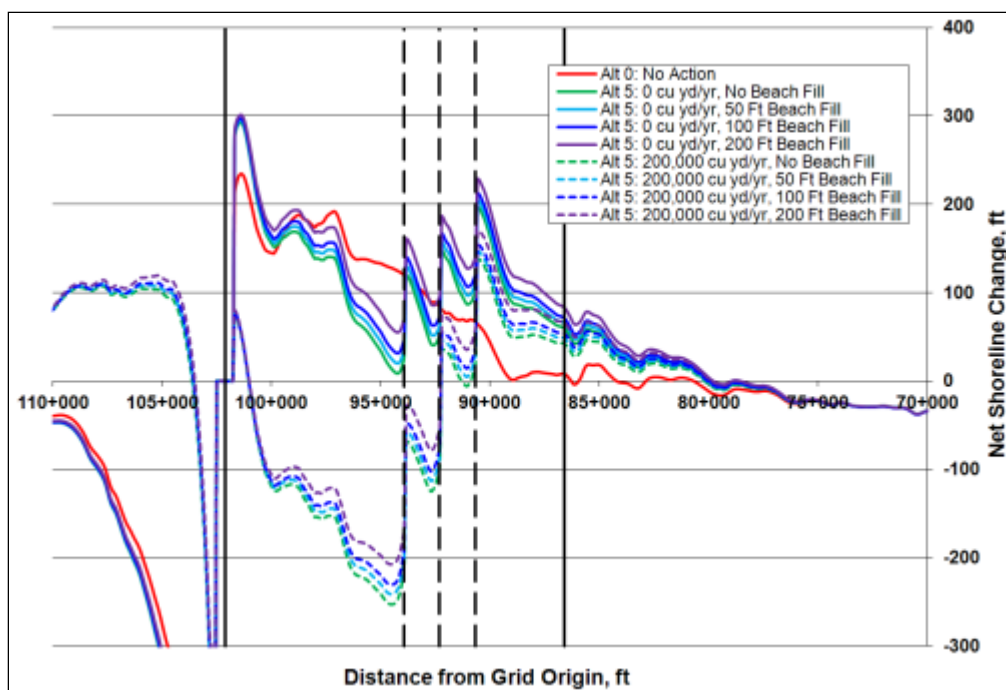
200,000 yd³/yr of bypassing MCR (Kraus et al. 2008). Design of the groin system was based on data collected before and immediately after construction of the new jetty at MCR. The proposed groin field will be dependent on the new equilibrium of MCR with the new east jetty. Therefore, it is imperative that the system be monitored until it has reached a new state of dynamic equilibrium (at least 5 yr). Only after that state has been achieved and data have been collected to re-evaluate the proposed system should this alternative be carried forward to the final design and construction phases of work.

Keep in mind, the recommended groin system design will advance the shoreline; however, without dune and vegetation management, it will not create a wider dry beach. Since the existing beach is presumably in equilibrium, the dune and vegetation will advance with the shoreline, maintaining the dry beach width. As the shoreline advances, management actions such as grading the upper beach will be required to widen the dry beach.

4.4.1 Environmental considerations

These would include increased turbidity during construction of the groins and beach. Beach maintenance operations have associated environmental

concerns that could be addressed through permits for ongoing beach maintenance. The additional beach will provide positive environmental benefits such as a turtle nesting habitat.

4.4.2 Habitat protected

Benthic habitat would be created from the construction of the groins. The wider beach would provide dry beach habitat and increased protection for landward habitat.

4.4.3 Storm damage vulnerability

The groins would help provide a wider beach within the target area, therefore increasing storm damage protection. Careful management of sand in the region (e.g., dredging and placement of sediment on the west beach) will be required to prevent flanking of the MCR west jetty. If a beach fill is constructed, the beach likely will require repair to replace eroded sand after a storm. The groins might require repairs if design conditions are exceeded.

For an engineered project to be eligible for federal assistance after a catastrophic event, it must have a maintenance plan and the sponsor must have the means and intent to maintain the project according to the plan. The plan should include:

- Project description: introduction, purpose, and description of the design with reference to appropriate design documents.
- Maintenance plan: specified renourishment intervals and maintenance standards for project-related structures such as pedestrian walkways; project monitoring such as yearly surveys, sand conservation measures, and future sand sources; documentation of local ordinances and participation.
- Maintenance actions: up-to-date ongoing summary.
- Project funding: for the design and construction of the project as well as for all maintenance actions to date.

4.4.4 Constructability

The groin would be built from land, applying similar construction techniques used at MCR in 2010. Structure details have been modified to enable land-based construction. No substantial issues with project constructability have been identified.

4.4.5 Regional considerations and enhancement

Bypassing at MCR reduces sediment trapped in the groin field but reduces erosion west of MCR, while a lack of bypassing has the opposite effect. This system is critically dependent on dredging operations and natural bypassing at MCR.

4.5   Preliminary groin system design

The GenCade model was applied to determine the groin length, location, orientation, and required fill, and the system's predicted performance. This section documents its design.

4.5.1 Armor size

Due to the hydraulic instability of rubble-mound structures, it is necessary to use empirical formulas to calculate the stone weight required to withstand design waves. The Hudson and Van der Meer equations were applied to determine the median weight of an individual armor unit in the primary cover layer, W_{50}, although only the Hudson equation is presented because it controlled stone size.

4.5.1.1   Wave transformation

To determine waves acting on the structure, they were transformed using CMS-Wave from the 66-ft depth contour to 12 ft, the depth immediately seaward of the groin toe. Nearshore waves at the groin were calculated as a function of water level, from -1.9 to 7.7 ft (NAVD88) and the 50-yr return period, for the offshore waves. Battjes and Groenendijk's (2000) formula was used to determine the statistical wave heights using as input the zero-moment wave height (H_{mo}), nearshore slope, and water depth. This method was applied to determine the 1% exceedance wave height, $H_{1\%}$, for input into the Hudson equation and significant wave height, H_s, for the Van der Meer equation. Figure 23 plots the nearshore $H_{1\%}$ and H_s as a function of water level for the offshore waves.

4.5.1.2   Stone stability

Hudson conducted investigations in the 1950s and early 1960s at the US Army Engineer Waterways Experiment Station (WES) and developed a formula to determine the stability of armor stone on rubble-mound structures (USACE 2002):

Figure 23. $H_{1\%}$ and H_s at the groin toe as a function of storm surge.
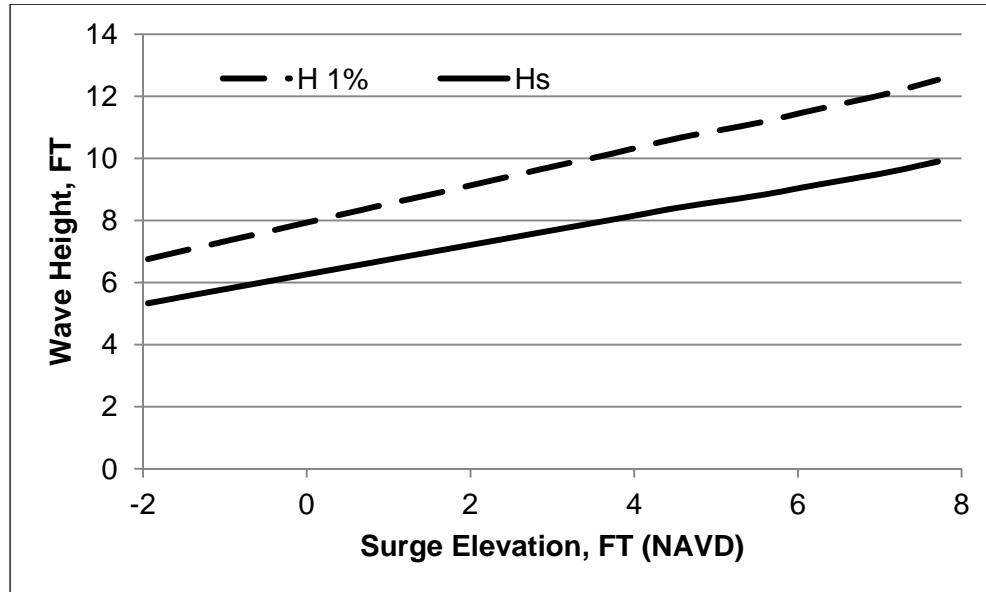
$$W_{50} = \frac{\gamma_s H^3}{K_D \left(\dfrac{\gamma_s}{\gamma_w} - 1\right)^3 \cot(\alpha)} \qquad (1)$$

where

γ_s = unit weight of the stone
H = design wave height
K_D = empirically determined stability coefficient
γ_w = unit weight of water
$\cot(\alpha)$ = slope of the structure

The unit weight of the stone was assumed to be 165 lb/cf (typical unit weight of granite armor stone used for placement in Texas) with side slopes of 3H:1V. A stability coefficient of 2 was selected, based on USACE (2002) and application of similar values for the recently constructed jetty at MCR (Kraus et al. 2008).

Specially placed rectangular-shaped armor stone is recommended, similar to most other nearby stone structures on the Gulf Coast, including the new jetty at MCR and the revetment at Sargent Beach. The $H_{1\%}$ wave heights calculated from Battjes and Groenendijk (2000) in Figure 23 were applied as the design wave height, adding conservatism. Figure 24 plots the cover stone weight vs. the surge elevation, neglecting reductions from overtopping or submergence.

Figure 24. Armor stone weight calculated with the Hudson equation.
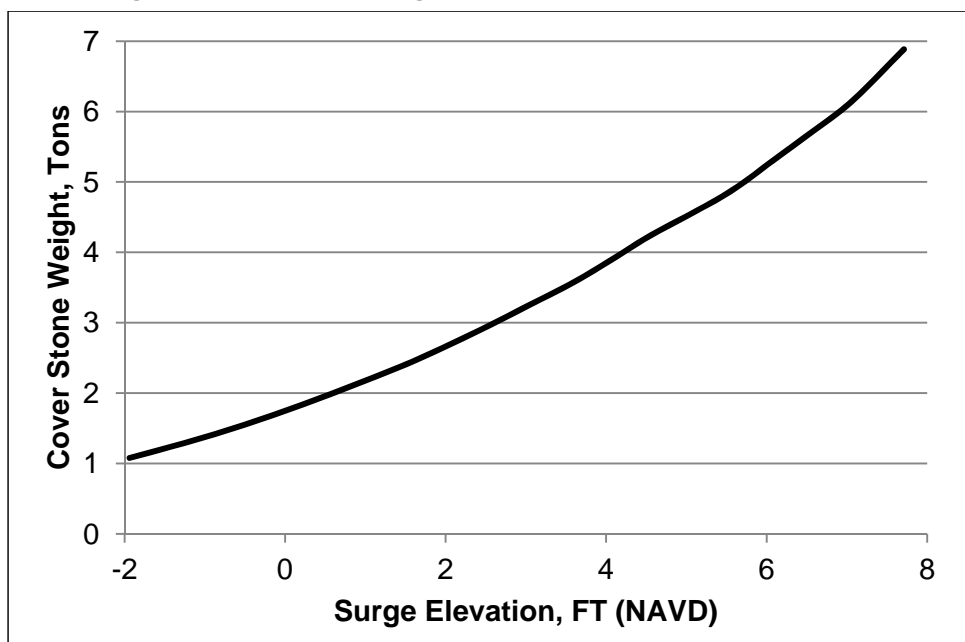
The recommended median armor stone weight is 8 tons: stable during a storm with 8-ft surge and 12-ft waves, approximately a 50-yr return period event. Although the calculations suggest that a 7-ton median armor stone would be sufficient, the heavier 8-ton stone was selected based on the design of nearby structures. This assumption helps to ensure more detailed analysis in the design phases of work will not result in a substantially higher cost if heavier stone is selected. A single layer of armor is recommended for the trunk and head to reduce total excavation requirements and stone quantities. This cost saving design feature should be investigated in more detail in final design. Armor stone weight was checked using the Van der Meer equation (USACE 2002), taking reductions for overtopping. Since the method resulted in a lower estimate, it is excluded from this report for brevity. It is possible to reduce the total structure cost through implementation of allowable reductions during final design or value engineering phases of work.

4.5.2 Suggested groin section

The groin system consists of three groins and a beach fill. This section documents selection of key design parameters and presents the recommended cross sections. Typical profile and section views are shown in Appendix D (Figure D2). Figure 25 shows a simplified sketch of the recommended groin cross section.

Figure 25. Preliminary groin design cross section.
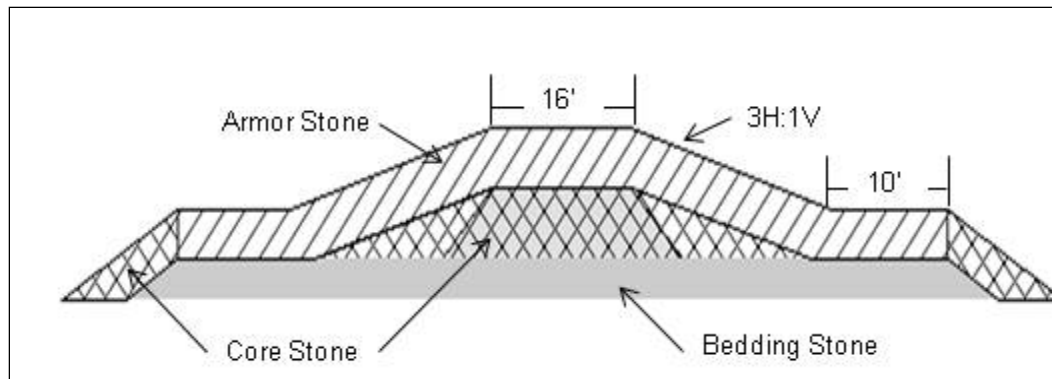
4.5.2.1   Groin crest and length

Each groin consists of a horizontal shore section (HSS), an intermediate sloped section (ISS), and an outer section (OS), plotted in Figures 26 and D2. Elevation of the HSS is set to approximately match the existing berm crest based on recent survey data, 4- ft NAVD. The elevation of the OS is typically set to Mean Lower Low Water (MLLW) to increase sediment bypassing and minimize stone. To help offset the influence of RSLR over the 50-yr design life, the elevation is set 0.5 ft above MLLW (0.0 ft NAVD). If RSLR continues at its historic pace, the crest of the OS will be 0.3 ft below MLLW in 50 yr. This will reduce slightly the trapping efficiency of the groins. If greater RSLR than the historical rate occurs, then the structure crest might need to be raised. Monitoring should be conducted.

The overall length of the groin was determined through analysis with GenCade. The length of the HSS is based on the desired maximum added beach width of 200 ft and the calculated maximum potential erosion on the downdrift side of the groins of 180 ft. Then the length of the ISS was set by following the natural beach slope of about 40H:1V to the +0.0 ft NAVD contour. Finally, the length of the OS is the total length determined by GenCade minus the length of the ISS and HSS.

4.5.2.2   Crest width

The minimum crest width is a function of the stone size, $D_{50} = 4.6$ ft. To improve stability and enable land-based construction techniques, the crest width should be specified with a 16-ft minimum, also ensuring the crest is at least three armor units wide.

Figure 26. Typical cross-shore profile of groin.
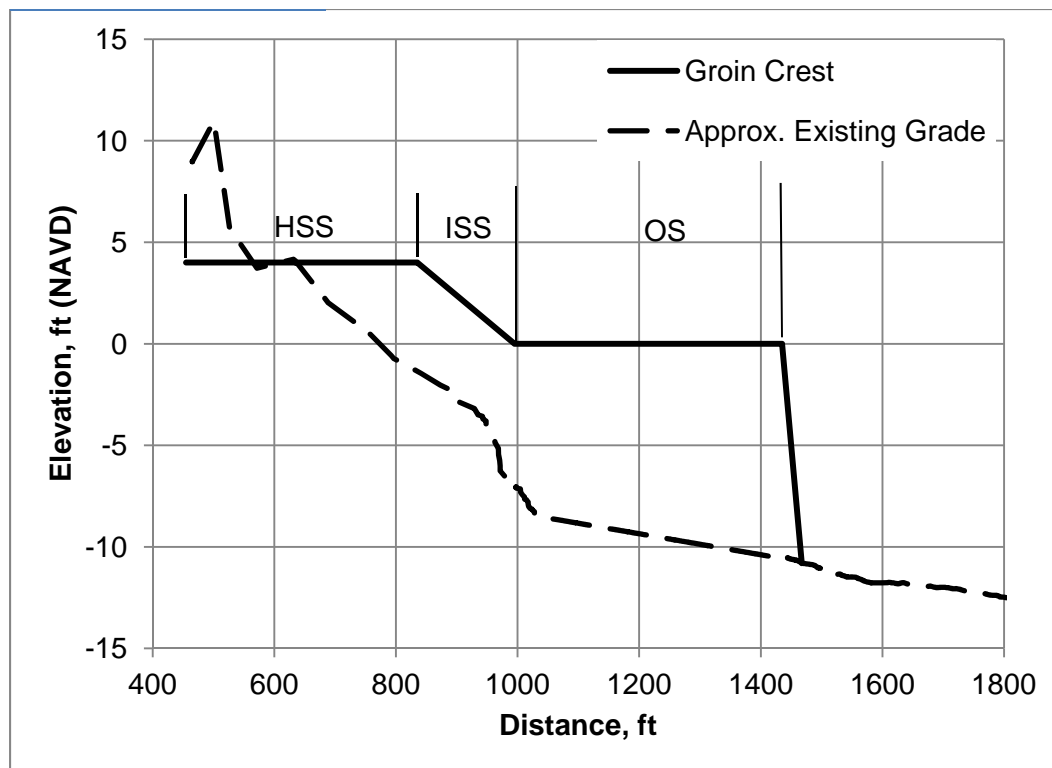
4.5.2.3   Scour protection

Detailed scour calculations have not been conducted for this level of design. A standard toe is shown in the details. Toe design should be verified during final design.

4.5.2.4   Side slopes

All side slopes were set at 3H:1V. Steeper side slopes are allowable and can reduce stone cost but will result in larger required armor stone. The flatter side slope was chosen to minimize required armor size and to reduce overall bid costs by making the structures easier to build.

4.5.2.5   Typical section details

Armor stone should be specially cut rectangular stone with 4- ft maximum thickness vertically to limit total section thickness. The structure should include a core to underlay the armor with median weight of about 1/10 the armor weight or about 1,600 lb. Core stone of weight ranging from 200 to 2,000 lb should be used, a common weight used in nearby USACE projects

(Kraus et al. 2008). The core should be chinked with filler stone under the crest to stabilize the armor and to facilitate land-based construction.

The core should be placed on a bedding layer with median weight ranging from about 1/200 to 1/4,000 of the armor weight. Galveston District, USACE, typically specifies ½-in. to 200 lb bedding stone to satisfy these requirements. A 3-ft thick bedding layer is recommended. A scour hole might form during placement, requiring more fill than shown in the neat line approximations.

The toe should be 2 to 3 times the armor layer thickness, with core stone placed at the end with approximately 1.5H:1V side slopes. Since the water depth is shallow relative to the waves, the armor should be placed to the bottom and used for the toe.

4.5.3   Beach fill design

A beach fill is optional in addition to the construction of the three groins. GenCade simulations were run with a 50-ft, 100-ft, and 200-ft wide beach fill. It is assumed that the grain size of the available fill will be the same as the native sand. However, the sediment source might not have the same grain size as the native sand. In the event the source sediment has a smaller grain size, a greater volume is necessary to create the same beach width. Conversely, a greater fill-grain size requires less volume to create the same beach width.

The construction template and design profile for the 100-ft wide beach fill is shown in Figure 27. More detailed construction templates for the 50-, 100-, and 200-ft wide beach fills are located in Appendix D. All three construction templates have a berm elevation of 4 ft (NAVD) and a slope of 1:30. Additional details about the beach fills, including volumes and berm width of the construction template, are located in Table 8.

The 200-ft wide beach was simulated in the cases shown in the previous section. After 5 yr, the cases with the beach fill advanced to 100 ft more than the cases without the beach fill. After 16 yr, the cases with the beach fills continue to protect the entire domain between 3 Mile Cut and MCR. The maximum amount of shoreline difference between the cases with and without the beach fills is about 25 ft. Although most of the protection provided by the beach fill is lost after 16 yr, there is still greater shoreline advance than the cases without the beach fill. Based on the 5- and 16-yr

Figure 27. Construction template and design profile for 100-ft beach fill.
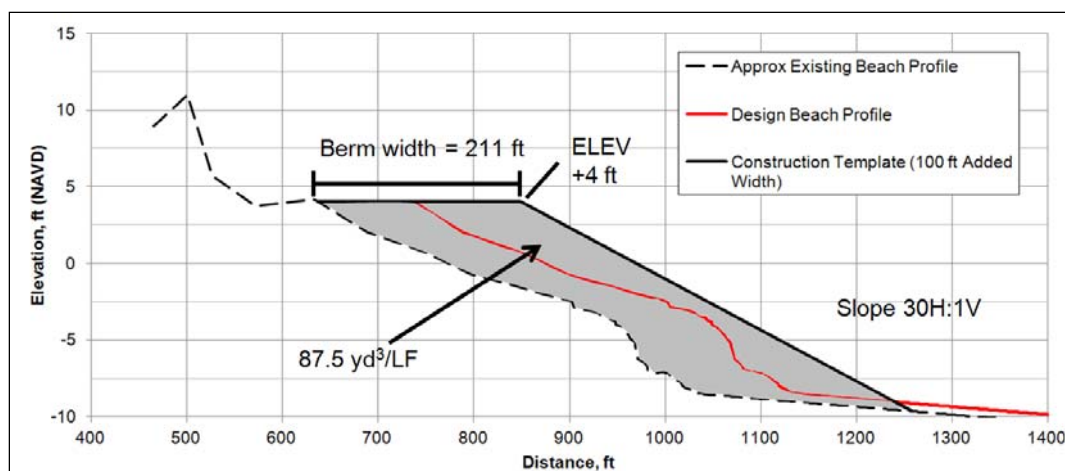
Table 8. Beach fill details.

Added Berm Width (ft)	Volume (yd³)	Nourishment Density (yd³ yd/lf)	Construction Template Details		
			Berm Elevation (ft NAVD)	Berm Width (ft)	Water Depth (ft NAVD)
50	143,750	44	4.0	125	-9.1
100	287,500	88	4.0	211	-9.7
200	575,000	175	4.0	379	-10.7

simulations, it is unlikely that any sand placed on the beach would remain after 20 yr. The frequency of renourishment depends on the rate of yearly bypassing at MCR. Due to additional erosion southwest of the last groin when bypassing occurs, it becomes necessary to renourish the beach more frequently than when bypassing does not occur. If the rate of bypassing is 200,000 yd³/yr, more than 150 ft of erosion occurs just southwest of the last groin between year 5 and year 16. Therefore, renourishment is recommended at least every 10 yr; the actual interval must be determined through monitoring. If no mechanical bypassing takes place, the entire shoreline from 3 Mile Cut to MCR advances from the initial shoreline after 16 yr. In this case, renourishment might not be needed for 20 yr. Finally, the renourishment interval also depends on monitoring and environmental conditions.

4.6 Preliminary opinion of probable construction cost

A preliminary opinion of construction cost (Table 9) was prepared based on the design presented, discussion with SWG engineers, and previous USACE projects. This preliminary opinion is intended to be used to help

the Port of Bay City Authority determine how to proceed with funding and constructing a shoreline stabilization project. The opinion should be updated during final design when more accurate estimates of material costs and quantities are available.

Table 9. MCR groin system preliminary opinion of cost.

ITEM	QUANTITY	UNIT	UNIT PRICE		EXTENSION	
Mobilization / Demobilization	1	LS	$	400,000	$	400,000
Topo / Hydro Acceptance Surveys	1	LS	$	75,000	$	75,000
Hazard (Magnetometer) Survey	1	LS	$	30,000	$	30,000
			CONTINGENCIES (30%):		$	151,500
			SUBTOTAL:		$	656,500
GROIN CONSRUCTION						
Armor Stone	34,296	Ton	$/Ton	120	$	4,115,520
Core Stone	24,363	Ton	$/Ton	90	$	2,192,670
Blanket Stone	32,496	Ton	$/Ton	70	$	2,274,720
Filler Stone	3,738	Ton	$/Ton	70	$	261,660
Foundation Excavation	37,338	CY	$/CY	4.0	$	149,352
			CONTINGENCIES (30%):		$	2,698,177
			SUBTOTAL:		$	11,692,099
			TOTAL W/O BEACH FILL:		$	12,348,600
COST PER 50 FT BEACH FILL						
Beach Fill	143,750	CY	$/CY	3.0	$	431,250
Transport and Install	143,750	CY	$/CY	10.7	$	1,538,125
Grading/Dressing Beach	143,750	CY	$/CY	4.0	$	575,000
Site Restoration	1	LS	$	15,000	$	15,000
			CONTINGENCIES (30%):		$	767,800
			SUBTOTAL PER 50 FT BEACH FILL:		$	3,327,200
			TOTAL W/ 50 FT FILL:		$	15,675,800
			TOTAL W/ 100 FT FILL:		$	19,003,000
			TOTAL W/ 200 FT FILL:		$	25,657,400

Notes
1. Cost of stone based on highest estimate from SWG planning guidance (Regner 2012).
2. Quantities and unit prices are based on approximate in place dimensions.
3. This Opinion of Construction Cost is based on data available at the date of publication and is not necessarily all inclusive of cost items. Actual construction costs may vary based on changes in market conditions.
4. Cost of beach fill based on Coastal Tech (2010) truck haul estimate. Cost of sand provided by Mike Griffith.
5. Cost includes construction of 3 groins and optional beach fill.
6. Project performance is directly related to volume of beach fill; reduced beach fill options, reduce performance.

5   Sargent Beach

This chapter describes the process to select a structural alternative near Sargent Beach. GenCade and CMS-Wave modeling results are discussed. A preliminary breakwater system design is shown and a preliminary opinion of construction cost is given.

5.1   Previous work

The goal on Sargent Beach is to halt erosion over the target area, shown in Figure 1. The Phase 1 study model results indicated that a breakwater field would best meet this goal given the presence of cohesive sediments and low net transport. After presentation of the Phase 1 study results to the Port of Bay City Authority, the final project goals were specified:

- Segmented breakwaters will be analyzed for shore protection at Sargent;
- The final alternatives might include a terminal groin;
- Beach fill can be included if necessary;
- An adaptive and scalable plan will be developed. The plan will ensure that the project functions as intended with minimal downdrift impacts. The plan will include multiple layouts to accommodate incremental funding;
- The first phase of the adaptive plan should consist of approximately 10 breakwater segments starting near Mitchell's Cut, moving northward/eastward.

5.2   Breakwater design overview

This section presents basic design criteria for segmented breakwaters with examples at other Gulf of Mexico projects given for comparison. Two design alternatives were considered in this section to help bind the number of cases evaluated with the numerical models in following sections. The first alternative encompasses barge-based construction, where a certain minimum depth is required. Alternative 2 assumes land-based construction techniques, where construction of the breakwaters via land-connected sand bridges is envisioned.

Due to similar wave climate and sediments, the work at Holly Beach,
Louisiana, is considered a reference for design. The primary design
objective, based on lessons learned at Holly Beach and other sites, would be
to reduce the longshore transport on the beach by about one half. A widely
used equation to calculate sediment transport is the CERC formula, where
the breaker height, H_b, is raised to the 2.5 power. Therefore, all things being
equal, a 50 % reduction in longshore sediment transport would require H_b
to be about 75 % of its original value, which coincides with the Holly Beach
breakwaters and is similar to the Grand Isle breakwaters (Gravens and
Rosati 1994; Edwards and Namikas 2011). Table 10 compares key
parameters at Sargent Beach, Holly Beach, and Grand Isle. Based on the
similarities, some initial design alternatives are proposed.

Table 10. Breakwater project comparison.

	Sargent Beach	Holly Beach	Grand Isle
Sediments	D_{50} = 0.128 – .186mm From last survey	0.15mm	0.15mm
Waves	similar	similar	similar
Currents	Possibly significant, CMS modeling	Prevailing westward current	Currents from Baratria Pass
Tide Range	Similar	Similar	Similar
Breakwater contour	4-5 ft (8 ft modeled)	5 ft (Mann and Thomson 2003), 4ft (Dean 2001)	6 ft
Beach Slopes	1V:100H+ (Figure 2) 1V:200H offshore Straight and parallel contours for the length of the project	1V:80H (Mann and Thomson 2003)	1V:100H (Gravens and Rosati 1994)
Breakwater Height	4 ft (above NGVD)	4 ft (above NGVD)	4 ft (above NGVD)
Foundation design	n/a	Mattresses; geotextile + small stone	unknown

5.2.1 Breakwater position – distance offshore

Holly Beach was constructed by rock placed from barges. Because the
construction occurred over several years by different contractors,
structures were placed at varying distances from the shoreline. This
variation is said to be due to the minimum depth required by contractors'
varying equipment. (Mann et al. 2004). From available documentation,
this depth appears to be 1.8 m (6 ft) at most of the 85 constructed Holly
Beach breakwaters. Four feet was said to be the "depth of sand closure" by
Mann and Thomson (2003) for Holly Beach.

Figure 28 shows examples along Sargent Beach in which it appears that a 4-ft depth includes the majority of the profile subject to active movement under ordinary conditions. However, 6 ft would more accurately describe the maximum depth of active movement under ordinary conditions -- certainly for the center and west locations of the project. The depth of closure of 19.7 ft selected for the GenCade work includes storm conditions (Thomas and Dunkin 2012). Selecting a depth of 4 ft gives a typical shoreline to structure distance of about 150 ft for Sargent Beach, consistent with the approximate 100H:1V slopes. The offshore distance for a 5-ft depth is 250 ft. The distance from the breakwaters to the shoreline varies from about 700 to 300 ft at Holly Beach.

Figure 28. Selected transects along Sargent Beach, November 2011.
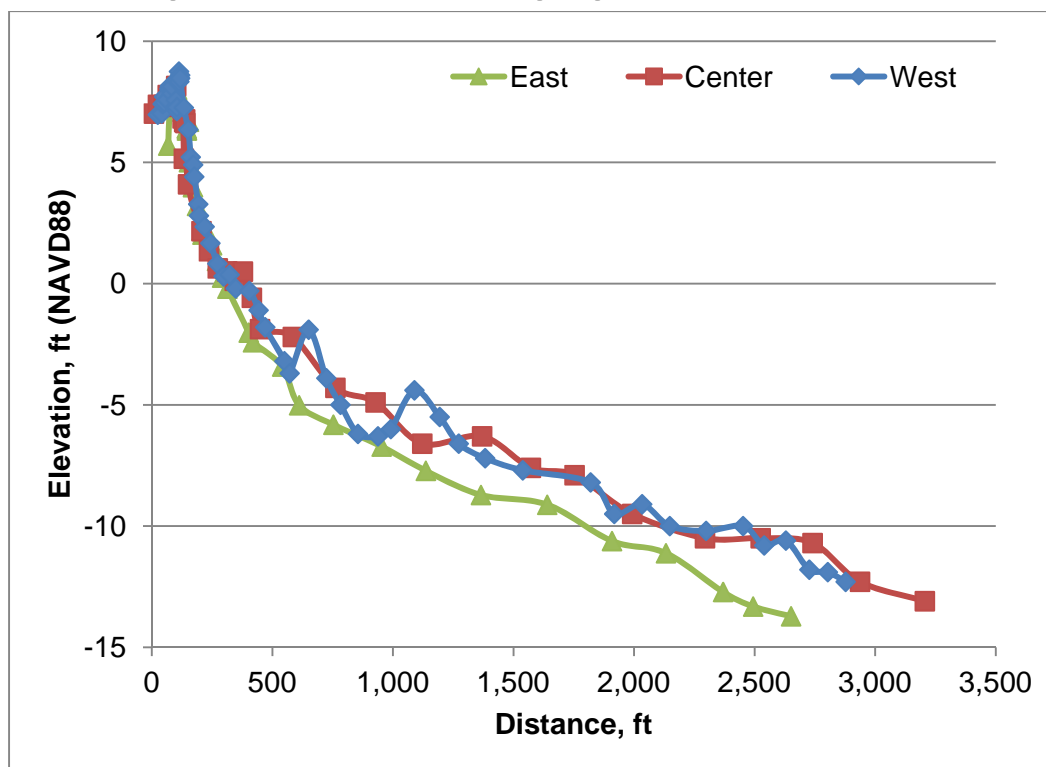
5.2.2 Breakwater crest height

Holly Beach and Grand Isle use a breakwater height of 4 ft above MSL. Mann et al. (2004) cautions that heights above this elevation could lead to structure-bearing pressure exceeding the soil-bearing capacity (not known at Sargent Beach but, due to the regional geology, requires a conservative value). Overtopping is related to breakwater height; lower heights result in increased wave-energy transmission and structural damage. Design waves are depth-limited at both Holly Beach and Grand Isle, and a crest height of

4 ft seems to have worked well in practice. Instances where the Holly Beach breakwaters were less than 4 ft resulted in increased wave transmission and reduced shore protection.

Note that quality control is essential during the construction process to assure the required breakwater heights are met. Periodic inspections should assure the height is maintained despite possible settlement from foundation soil consolidation or rock nesting. Most of the effects from these factors are expected to occur during or soon after construction, so incentives and mitigation should be provided in the construction contract.

5.2.3 Breakwater length

Important factors for breakwater design are the breakwater segment length (L_s) and the gap size (L_g). The greater the ratio of breakwater length is to gap width, the greater the project cost (the cost is assumed to vary linearly with total required breakwater length). Sufficient gaps should be included to enable the outflow of return currents due to wave mass transport. Also, breakwater length and gaps can be selected to enhance wave energy dissipation for a particular wave climate. The Holly Beach breakwaters, shown in Figure 29, have a length of about 180 ft and a gap width of 260 ft giving a L_s/L_g ratio of 0.7 (the distance to the shoreline [Y] is about 380 ft). Mann and Thomson (2003) reports that, after an evaluation, some break- water gaps were changed to 0.7. Gravens and Rosati (1994) reported that with the 200-ft segment length modeled at Grand Isle, a L_s/L_g ratio of 1 or greater gave essentially the same effect as a single continuous breakwater.

Figure 29. Holly Beach breakwater example.
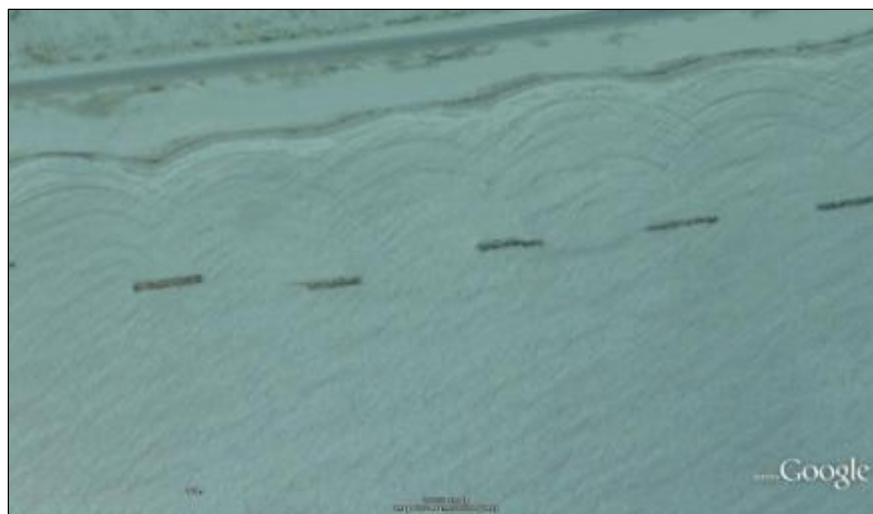
5.2.4 Minimum breakwater depth

The shoreline distance is determined from the desired breakwater depth. As discussed in Alternative 1, the minimum depth of sand closure is 4 ft. The goal is to allow the beach to respond to the wave energy without the breakwater affecting the cross-shore transport of sand or interfering with longshore currents. For a breakwater depth of 40 ft, the offshore distance is about 150 ft, and for 3 ft the offshore distance is approximately 80 ft.

5.2.5 Empirical breakwater design summary

Empirical parameters discussed in this section are summarized in Table 11. Based on guidance by various authors and successful nearby projects, the empirical parameters summarized in Table 11 were refined to determine an initial case to be modeled.

Table 11. Empirical parameter summary.

	Sargent Beach	Holly Beach (Post Rehab)	Grand Isle (modeled)
Breakwater height, ft	4	4	4
Breakwater depth, ft	5+	4-6	6
Distance from shoreline (Y ft)	350+	393-663 (Dean 2001) 1995 shoreline.	600
Ls(ft)	220	160 -240	200
Lg (ft)	315	240-330	300-350
Ls/Lg	0.7	0.5-0.8	0.57 – 0.6
Ls/Y	0.88 - 0.36	0.55 ,0.45 , 0.33	0.33
Y if Ls/Y = 0.333	660	480 -720	600
Y if Ls/Y = 0.125	1760	1280 -1920	1600
Ls	4.8	4.4, 4.6, 4.8	4.8
Kt (Overtopping% = 10)	63%	69-71%	64%

Dean (2001) gives the wave transmission parameter *Kt*, which is dependent on the breakwater dimensions

$$Kt = \frac{Lg}{Ls + Lg} + Ls \; {}^{*}\frac{Q}{Ls + Lg} \tag{2}$$

where Q = Percent overtopping.

Guidance from the Coastal Engineering Manual (CEM) (USACE 2002) suggests that L_s/Y should be less than (0.1250.33), where Y is the distance from the breakwater to the shoreline. These parameters are shown in Table 11. Holly Beach appears to be at the upper end of this guidance. The dimensionless parameter I_s represents the morphologic form of the shore response as subdued salients for $I_s = 4$ and no sinuosity for $I_s = 5$ and is calculated as:

$$Is = exp\left(1.72 - 0.41\frac{Ls}{Y}\right) \tag{3}$$

Gravens and Rosati (1994) notes that predictive formulas in the CEM do not adequately consider the Grand Isle (and consequently Holly Beach) site conditions, which include finer sediments and shallower slopes than the data upon which the formulas are based.

5.3   Numerical modeling of breakwater alternatives

Once the initial dimensions were selected, numerical models were applied to further refine the recommended design and to calculate performance. This section documents results of both the GenCade and CMS model applications for breakwater design. The most important model limitation common to both models is that the sediment transport equations represented in the models do not fully consider the properties of Sargent Beach cohesive sediments. To compound the uncertainty, detailed geotechnical properties and data are not available for this study. To address this limitation, model results are interpreted in a semi-qualitative and relative sense, building on analysis of nearby and similar projects and empirical equations.

5.3.1 Shoreline change modeling

Shoreline change modeling was conducted using GenCade to help refine breakwater parameters, to assess performance of the breakwater system, and to attempt to quantify adjacent impacts. During the course of this model application, GenCade limitations were exceeded. Application of the code to evaluate breakwaters is still in development. Therefore, these model results should be applied with caution. The results were used to support design based on empirical formula, adjacent projects, and CMS results.

5.3.1.1   Model setup

The setup for the GenCade alternatives at Sargent Beach was similar to the
setup used for the calibration and validation. The same units, coordinate
system, and datum were used; however, the model domain was shortened to
decrease the amount of time necessary to simulate each alternative. The
grid extends from about 5.1 miles southwest of the Brazos River to about
3.6 miles southwest of Mitchell's Cut for a total of 16 miles. The cells range
in size from 33 to 490 ft (10 to 150 m). The smallest cells were necessary to
provide detail in the breakwater regions and to meet GenCade model
guidance recommendations regarding the number of cells representing a
breakwater. The 33 ft-sized cells range from just to the northeast of the first
breakwater to just northeast of Mitchell's Cut. Each breakwater phase
recommends a different number of breakwaters; therefore, the GenCade
grid (Figure 30) for each phase is unique. For example, the first phase
includes the least number of breakwaters and only has 358 cells. The
smallest cells in the grid for Phase 1 extend about 1.1 miles (1.77 km). The
phase with all breakwaters simulated requires 1,411 cells since the smallest
cells must extend more than 8 miles (over 12.8 km).

Figure 30. GenCade setup between MCR and 3 Mile Cut for Alternative 0.
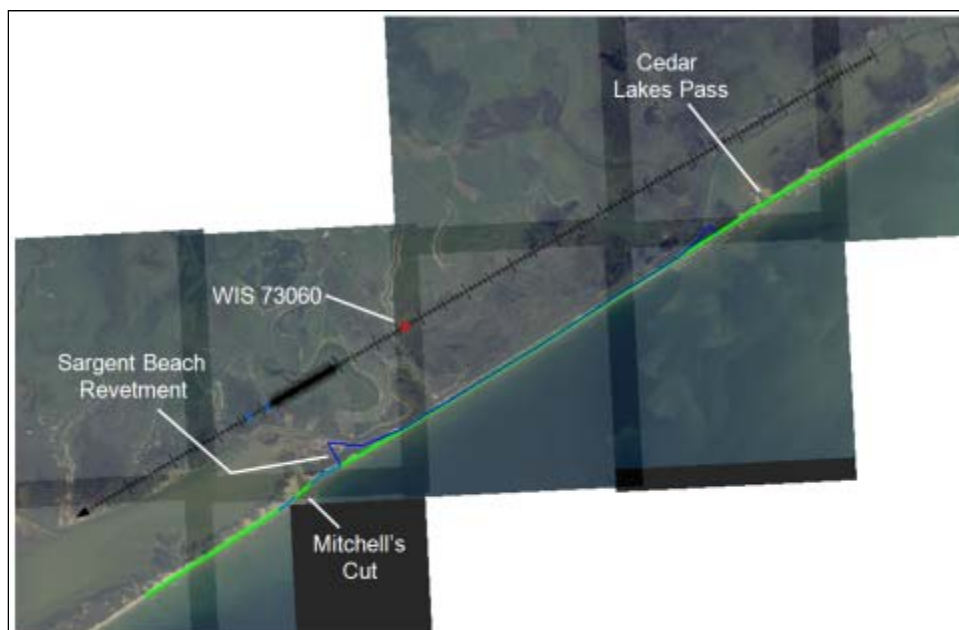
Because the grid is 16 miles long, only data from WIS hindcast station
73060 was applied. The same procedure for waves was conducted at
Sargent Beach as the analysis at MCR. All of the alternatives were
simulated for 5 yr while most were also simulated for 16 yr.

5.3.1.2 *Procedure and definition of alternatives*

Based on results of Section 5.2, a breakwater length of approximately 220 ft and a gap size of approximately 330 ft were initially modeled with GenCade. Sensitivity tests were conducted to help quantify model uncertainty. To begin, the breakwaters were placed at a distance of 400 ft for one test and at 800 ft offshore for a second test. For each set of tests, the transmission coefficient for all the breakwaters was adjusted.

Once it was determined that breakwaters nearer to the shore performed better, a second set of sensitivity tests were conducted for breakwaters at an offshore distance of 400 ft. Different profiles from the survey data show that the depth at 400 ft offshore varies. Therefore, additional sensitivity tests were conducted with breakwaters at 400 ft offshore at depths between 4 and 7.5 ft. Through the sensitivity tests, a transmission coefficient of 0.5 was chosen to produce the desired salient or tombolo for all subsequent alternatives in GenCade. While it is recommended that the breakwaters be constructed at a distance of 350 ft offshore, many alternatives were also simulated with breakwaters at a distance of 400 ft offshore. However, these results become repetitive, so details have been placed in Appendix A.

Following this initial sensitivity study, lengths of the breakwaters and gaps at the first and last breakwater were adjusted. By shortening the break-waters and lengthening the gap size between, it was expected that the end effects would be reduced. The first alternative decreased the length of the first and last breakwaters to 98 ft (30 m) and increased the first gap size to 460 ft (140 m). A second sensitivity test involved decreasing both the first two breakwaters and last two breakwaters. For a case with 10 breakwaters, a total of four breakwaters would be 98 ft long, and six would have the original length of 70 m (220 ft).

After sensitivity tests were conducted, several phases of construction were considered. For example, it is unlikely there would be enough funding to construct breakwaters in front of the entire revetment at Sargent Beach. Monitoring would need to take place after initial construction to determine if the project is performing as expected. It is suggested that the construction take place in phases. The first phase should include 10 breakwaters just to the north of Mitchell's Cut, at the downdrift end of the project area, so these structures would capture longshore transport before the next phase of construction. Following phases of construction would be based on monitoring results from the initial demonstration project. Another phase

(or alternative) modeled with GenCade included 15 breakwaters just to the north of Mitchell's Cut and 15 breakwaters at the northern end of the revetment. This alternative would determine project sensitivity to the construction sequence and examine erosion of the beach at the northern end of the project before construction could occur while allowing the center section of the revetment between the breakwater segments to become exposed. The largest breakwater field analyzed includes 81 breakwaters, covering all of Sargent Beach.

Because design of any full scale breakwater project would be based on data gathered after construction of the initial 10-segment demonstration project, only Phase 1 is discussed in the remainder of this section. Other potential phases of construction are included in Appendix A.

5.3.1.3   Alternative 0: No action

A baseline case with No Action was simulated to compare to each phase after 5 yr. This baseline case included Mitchell's Cut but did not include any structural alternatives. Figure 30 in the previous section shows the grid and initial setup. This case was run for 5 and 16 yr; only results after 5 yr are shown. Figure 31 shows the No Action case after 5 yr. In all of the following figures, the black line at 67+000 ft represents Mitchell's Cut. The black line extending from about 22+000 ft to about 64+000 ft represents the revetment.

If no action is taken, it is likely that the trend of erosion will continue. After 5 yr, the shoreline erodes to the revetment from 25+000 ft to 56+000 ft. Increased erosion adjacent to the revetment is possible since no sediment will be supplied by erosion of beaches at the revetment.

Figure 32 shows the gross transport in cubic yards averaged over the 5-yr period. The gross transport rate is fairly constant around 400,000 yd³/yr over the grid. The gross transport rate drops slightly from about 400,000 yd³/yr to about 370,000 yd³/yr at 25+000, which is the location where the shoreline intersects the revetment. The transport rate drops to 0.0 yd³/yr at Mitchell's Cut, because transport is not calculated within an inlet.

5.3.1.4   Phase 1: 10 Breakwaters

Ten breakwaters were modeled just north of Mitchell's Cut to represent the Phase 1 demonstration project. For the initial case, all 10 breakwaters were the same length and the same distance apart. Phase 1 was run for 5 and

Figure 31. Simulated shoreline change for Alternative 0: No action after 5 yr.
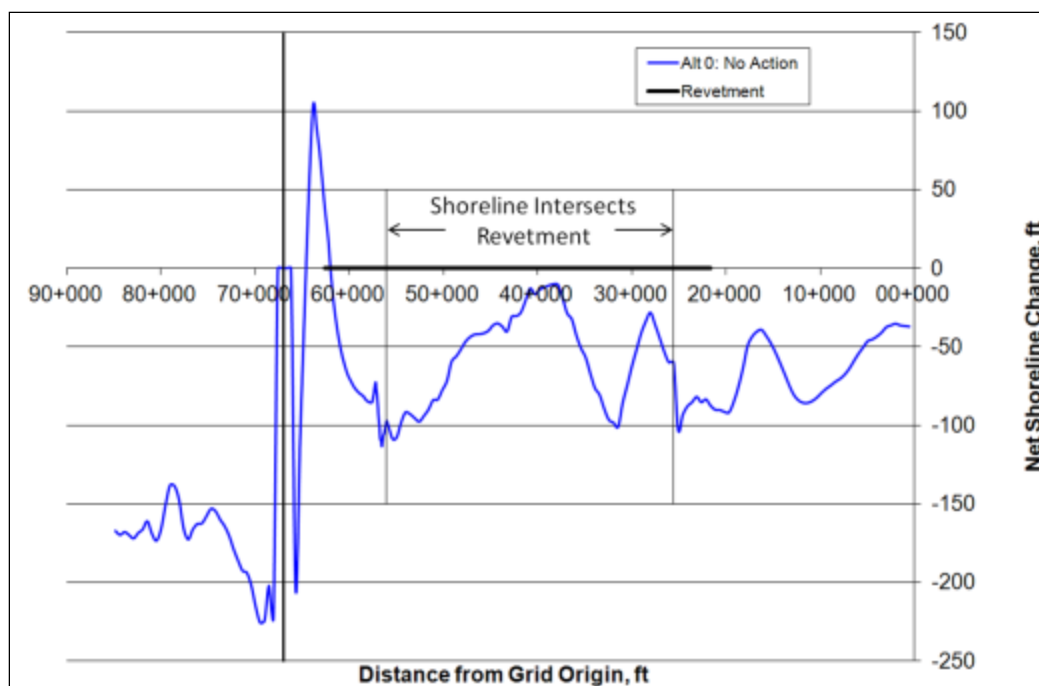
Figure 32. Average gross transport for 5 yr Alternative 0: No action simulation.
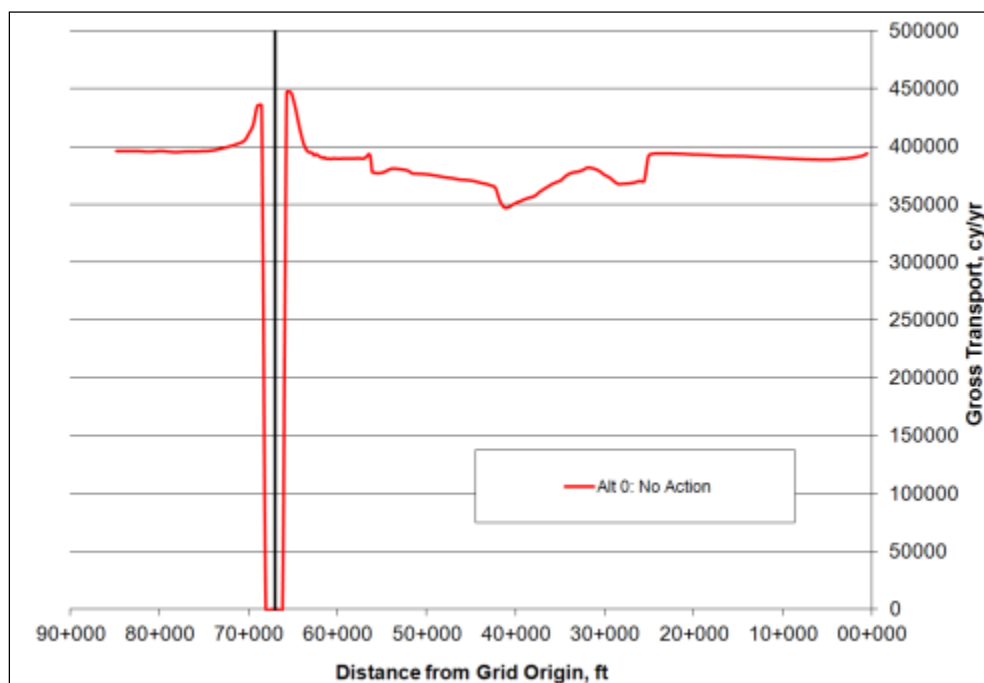
16 yr, but only the results after 5 yr will be discussed here. The first sensitivity tests for breakwaters in GenCade were conducted with break-waters 400 ft offshore. It was determined later in the study that breakwaters at a distance of 350 ft offshore were optimal. This section will compare the

results for the 350 ft and 400 ft offshore breakwaters. The setup in GenCade is shown in Figure 33. The orange lines just offshore and north of Mitchell's Cut represent the breakwaters.

Figure 33. GenCade setup for Phase 1 (10 breakwaters shown in orange).
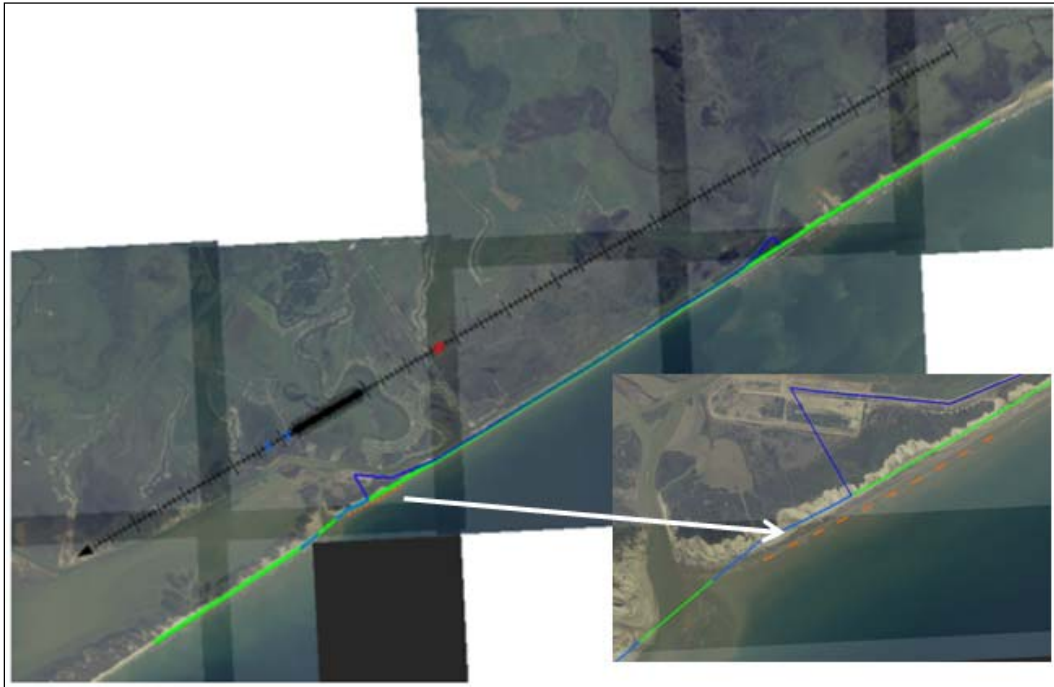
Figures 34 and 35 compare different lengths and gap sizes near each end of the breakwater field to constant breakwater lengths and spacing. In both figures, "Constant Spacing" represents the simulation in which all breakwaters were 220 ft long and all gap sizes were 330 ft. The first and last breakwaters were shortened to 98 ft while the gaps between those breakwaters were increased to 460 ft. Then the first, second, ninth, and 10th breakwaters were all shortened to 98 ft while the gaps between those breakwaters increased to a length of 460 ft. In the figures' legends, the simulations are represented by "Phase 1: Shorter Breakwater at each end" and "Phase 1: 2 Shorter Breakwaters at each end." Figure 34 shows the shoreline change for the simulations with breakwaters located at a distance of 350 ft offshore.

Regardless of the spacing of breakwaters at the ends, they reduce the erosion from about 52+000 ft to Mitchell's Cut. Tombolos form at some of the breakwaters, while salients occur at the others. The case with break-waters at a constant length and spacing results in tombolos at the ninth and 10th breakwaters. Reducing the length of the breakwaters near the ends

Figure 34. Shoreline change after 5 yr for Phase 1 breakwaters at 350 ft offshore with varying breakwater lengths.
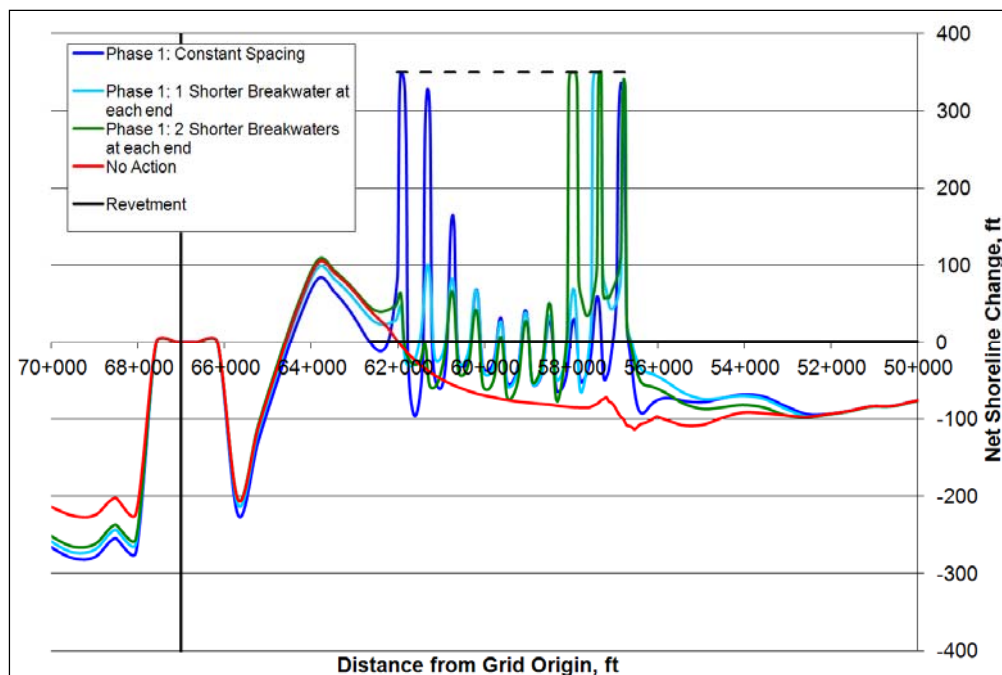
Figure 35. Shoreline change after 5 yr for Phase 1 breakwaters at 400 ft offshore with varying breakwater lengths.
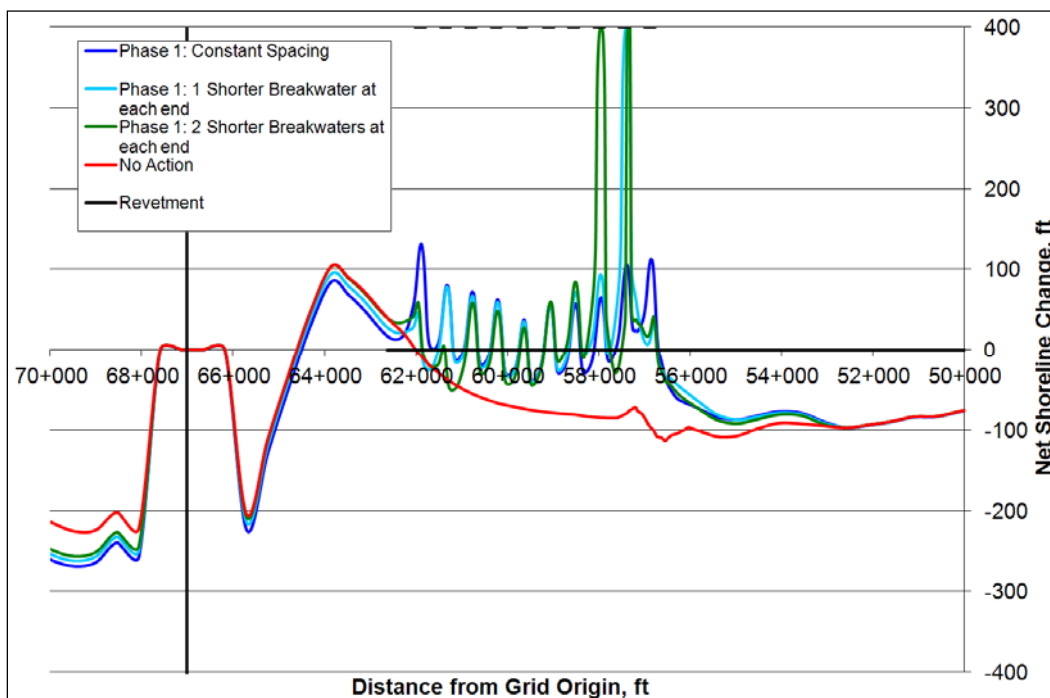
removes the tombolos behind the breakwaters directly northeast of Mitchell's Cut. A tombolo forms at the first breakwater when constant length breakwaters and spacing is used. A tombolo also occurs behind the

second breakwater when the first breakwater is shortened. When break-
waters are present in the model, additional erosion occurs to the south of
Mitchell's Cut. These cases do include a 36,000 yd^3/yr mitigation beach fill
directly to the south of Mitchell's Cut. The beach fill volume was calculated
based on model results using waves of constant height, period, and direc-
tion. The average wave height of all wave events in the 1995-2000 WIS
hindcast was used as the constant wave height. The WIS waves were used
for the cases shown in Figure 34. Since large waves from storm events are
included in the WIS, it makes sense that a different beach fill volume is
necessary to mitigate for additional erosion south of Mitchell's Cut. In these
cases, the fill does not completely negate the added erosion but helps
control it.

Figure 35 is very similar to Figure 34; however, the breakwaters have been
moved to an offshore distance of 400 ft. For this case, no tombolos form
after 5 yr when the breakwaters are a constant length and distance from
each other. Tombolos form behind the northern breakwaters when the
breakwater lengths are reduced and the space between is increased. While
the beach behind the breakwaters does not widen, it does not erode as
much as the No Action alternative. Again, the breakwaters cause
additional erosion to the southwest of Mitchell's Cut.

When Phase 1 was compared to Alternative 0: No action, Phase 1 caused
much more erosion to the southwest of Mitchell's Cut. While this study will
not detail a beach fill southwest of Mitchell's Cut for mitigation, Phase 1 was
simulated in GenCade with and without a 36,000 yd^3/yr beach fill. These
cases are compared with the Alternative 0: No Action cases in Figures 36
and 37. While the beach fill reduces erosion to the southwest of Mitchell's
Cut, it does not eliminate the additional erosion caused by the breakwaters.

The case with breakwaters at 400 ft offshore is also compared to the case
with breakwaters located at a distance of 350 ft offshore (Figure 38). The
case in which breakwaters are 400 ft offshore results in slightly less
erosion to the south of Mitchell's Cut; however, both cases experience
increased erosion compared to Alternative 0: No Action. When the
breakwaters are closer to shore, it is more likely tombolos will occur,
especially behind the breakwaters near the ends. Both cases reduce the
erosion behind the breakwaters compared to Alternative 0: No Action.

Figure 36. Shoreline change after 5 yr for Phase 1 breakwaters at 350 ft offshore and with and without beach fill.
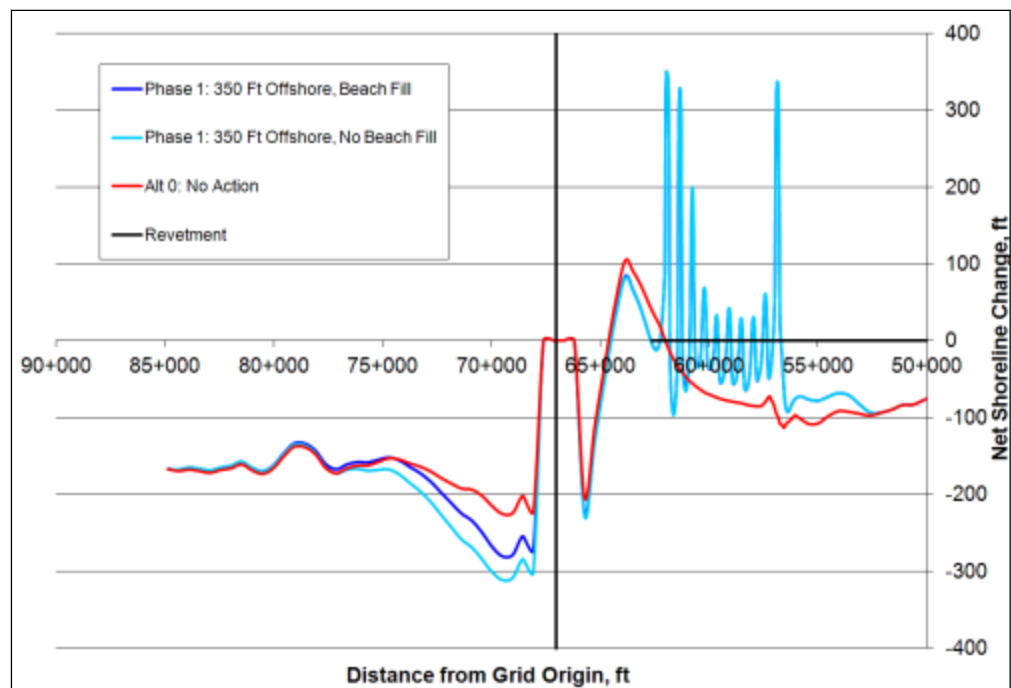
Figure 37. Shoreline change after 5 yr for Phase 1 breakwaters at 400 ft offshore and with and without beach fill.
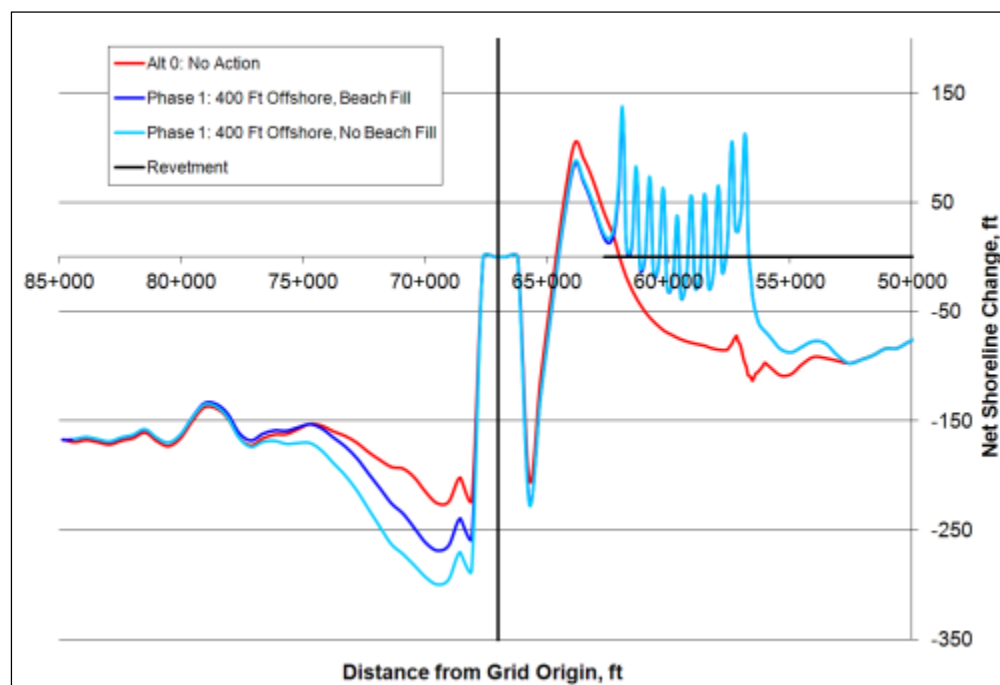
Figure 38. Shoreline change after 5 yr for Phase 1 breakwaters at 400 ft and 350 ft offshore.
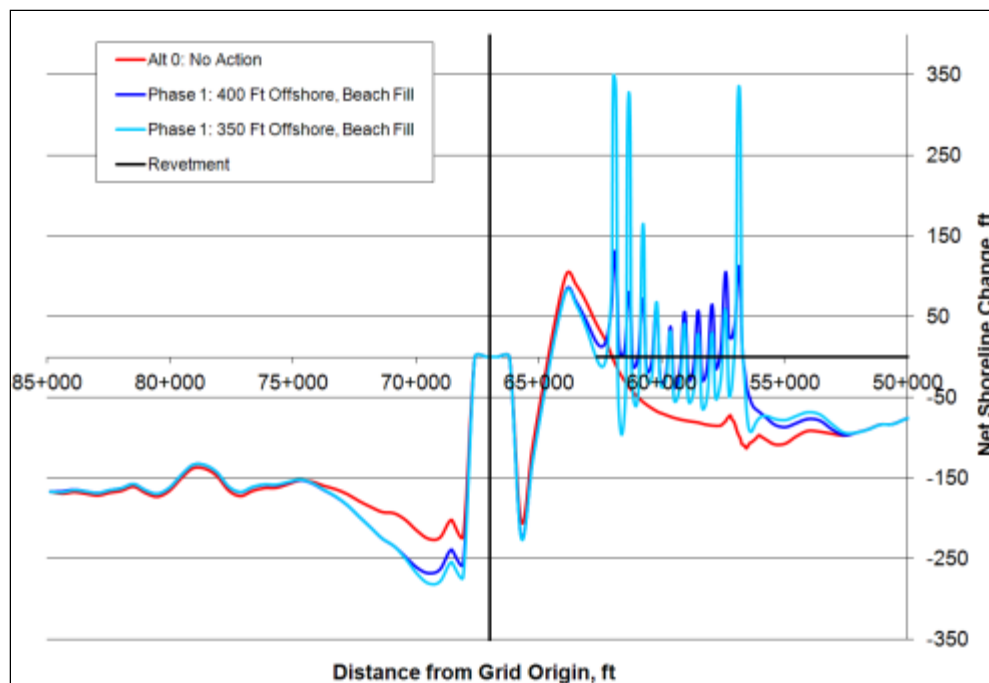
The gross transport averaged over 5 yr is shown in Figures 39 and 40. The figures compare the gross transport rates for each breakwater configuration at a distance of 350 ft offshore. When breakwaters are not present, the gross transport rate remains about 400,000 yd³/yr near Mitchell's Cut. The gross transport rate for all three of the different breakwater configurations decreases to about 225,000 yd³/yr. The tombolos that formed behind the first and second breakwaters when those breakwaters were shortened also affect the gross transport. Figure 40 provides a close up of the gross transport rates in the vicinity of the breakwaters.

Figure 41 compares the constant-length breakwaters at 350 ft and 400 ft offshore. The distance offshore makes little difference in the gross transport rate at the breakwaters. Both cases reduce the gross transport rate about 45 % from about 400,000 yd³/yr to about 225,000 yd³/yr.

The layout of the selected demonstration project plan is shown in Appendix D (Figure D3). The plan is based in part on these model results. Since adjusting the breakwater length and gap size near the ends of the breakwater field had little impact on the gross transport, a constant breakwater length and gap size was used. Although other phases of con-struction (up to complete protection) for Sargent Beach are discussed in Appendix A, no larger project should be constructed until the

demonstration project has been built and monitored. The length of each breakwater and the gap size is the same in all phases discussed in Appendix A.

Figure 39. Average gross transport for 5 yr simulation with varying breakwater lengths.
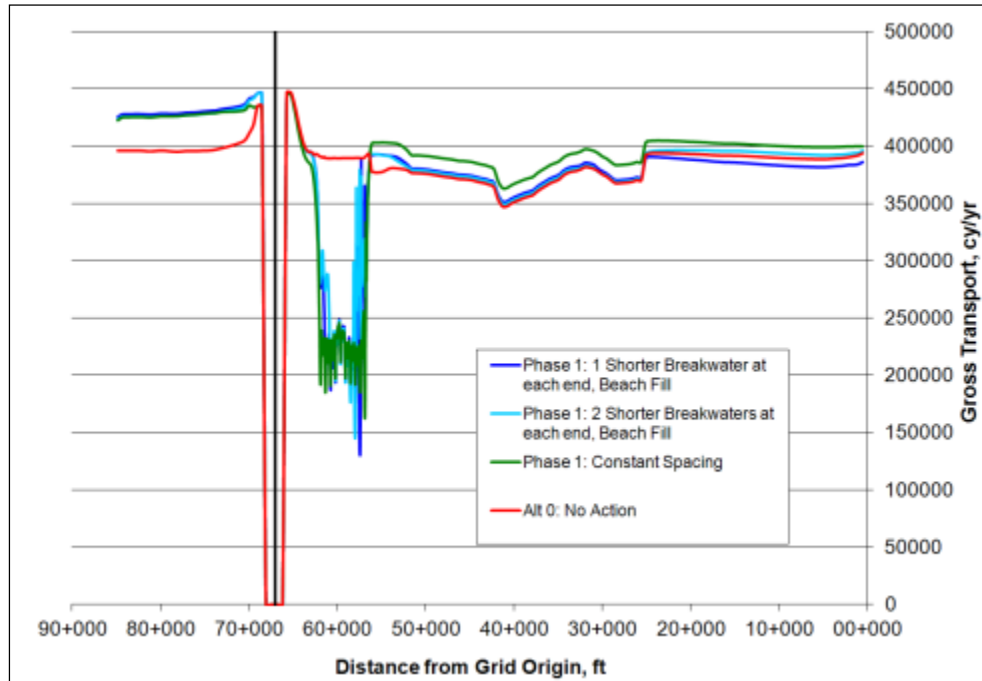
Figure 40. Average gross transport for 5 yr simulation with varying breakwater lengths with increased detail.
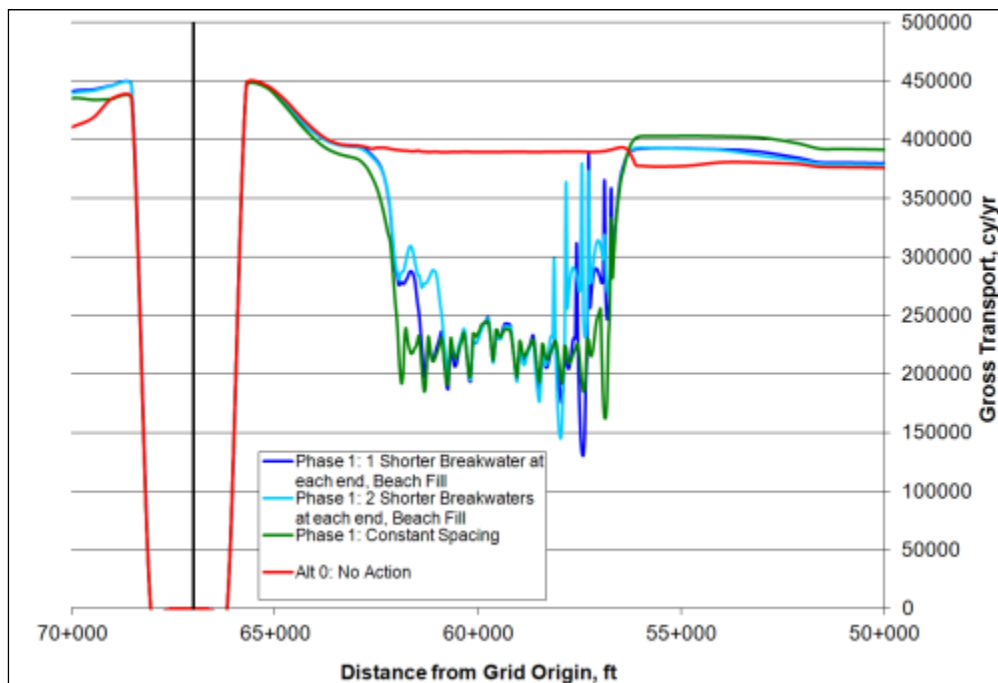
Figure 41. Average gross transport for 5 yr simulation for 350 ft and 400 ft offshore.
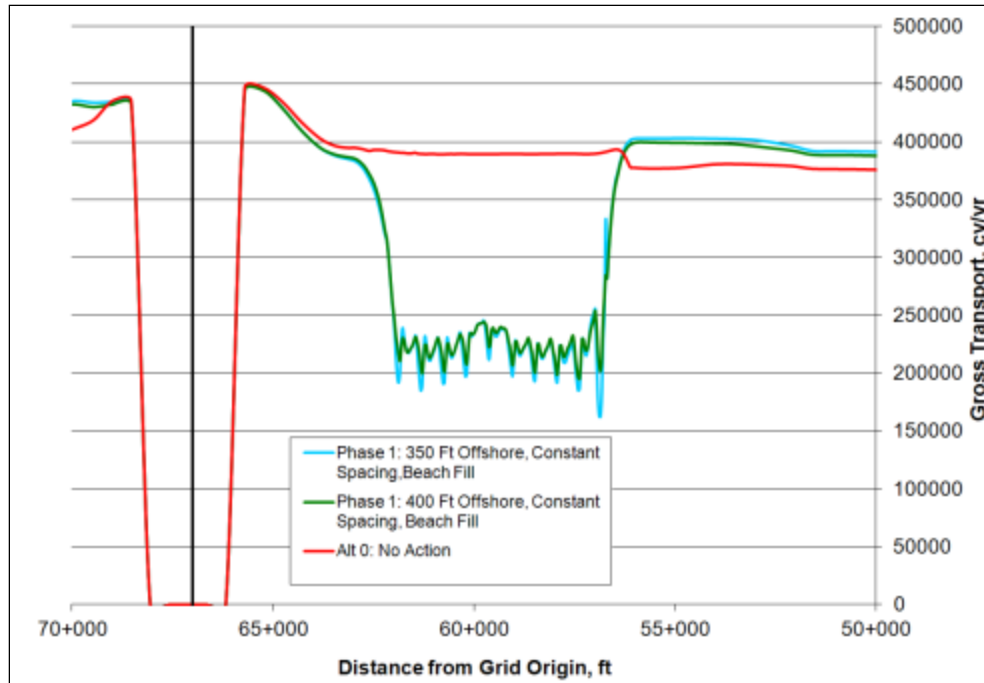
5.3.2 CMS modeling

Breakwater configurations were analyzed iteratively with the CMS to help refine design. This section includes specific relevant engineering results obtained through analysis in CMS. Appendix B includes more detail on CMS modeling. One CMS limitation of note is that swash zone transport processes are not included. Like the other analyses, CMS was applied in concert with additional techniques to enable successful project design.

5.3.2.1 Salient size

Hsu and Silvester (1990) give a relation:

$$X = Ls * 0.678 \left(\frac{Y}{Ls} \right)^{1.215} \qquad (4)$$

where

X = the distance from the salient tip to the breakwater;
Y = the shoreline to breakwater distance;
L_s = the breakwater length.

Their relation was developed for single breakwaters, but it can also be applied to multiple breakwaters. Using the project dimensions L_s = 220 ft and Y of 350 ft, X is calculated as 262 ft and the computed salient length Y_s is 88 ft (Note: $Y_s = Y\text{-}X$). Comparing this value to computed results is useful. Figure 42 shows CMS results computed for one month with 0.14 mm sand. In the figure, warmer colors (yellow, orange, red) represent accretion and cooler colors (blues) represent erosion. The calculation shows that accretion occurs behind each of the breakwaters. The arrow in the figure represents distance from salient tip to the breakwater of 90 ft. The GenCade shoreline response plots in the previous section show a salient size of about 90 ft for the middle breakwaters, verifying model consistency. The GenCade results were saved for each week of time simulated in the model. Though not shown, the CMS–GenCade comparison is consistent at the one-month CMS timescale.

Figure 42. CMS computed morphology change after 1 month run.
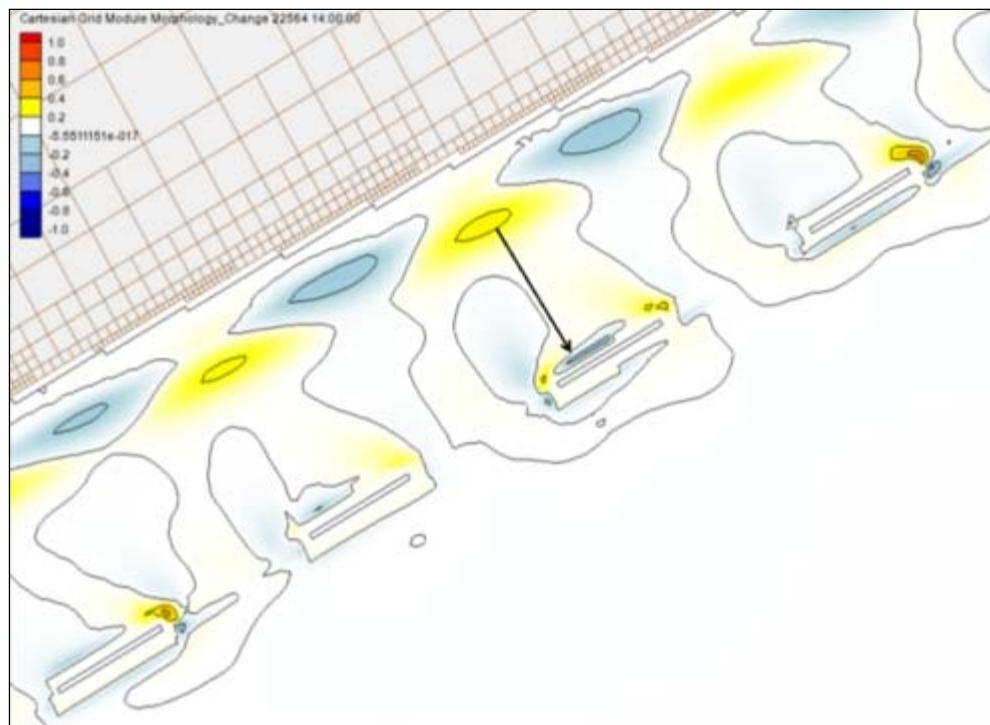
5.3.2.2   Transmitted waves

Table 11 uses an assumed overtopping rate, Q, of 10 % (Mann and Thomson 2003). Computed CMS results using the Table 11 Sargent Beach parameters were applied to verify this assumption. An average incident wave condition was applied within the model; breakwater incident and transmitted waves were recorded (Figure 43). The transmitted wave heights are shown as a

histogram in Figure 44. Transmitted wave heights greater than 1 ft (30 cm) are considered to represent overtopping. The observed overtopping rate for these average conditions was 13 %.

Figure 43. CMS observed breakwater wave heights incident and transmitted.
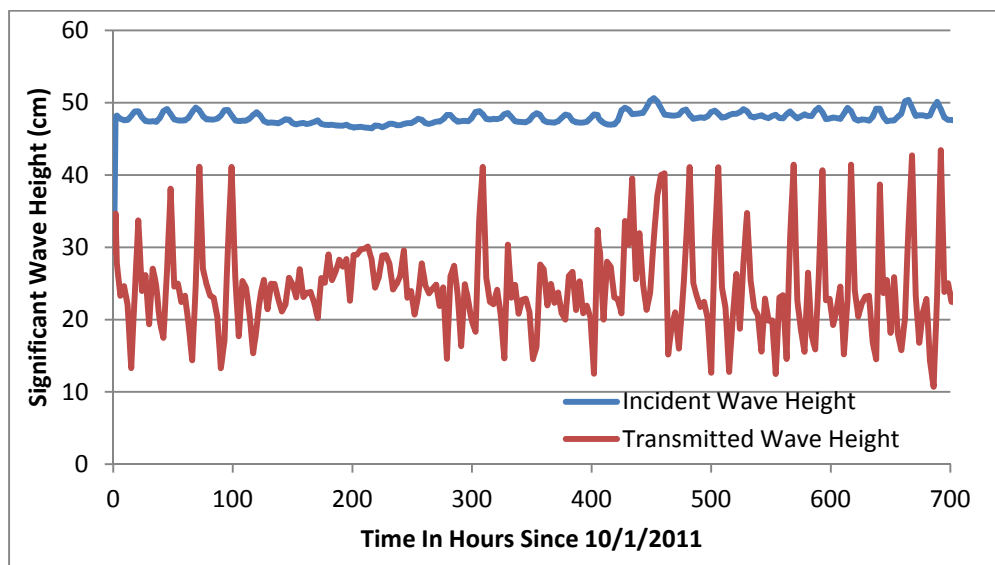
Figure 44. Histogram of transmitted wave height shown in Figure 43.
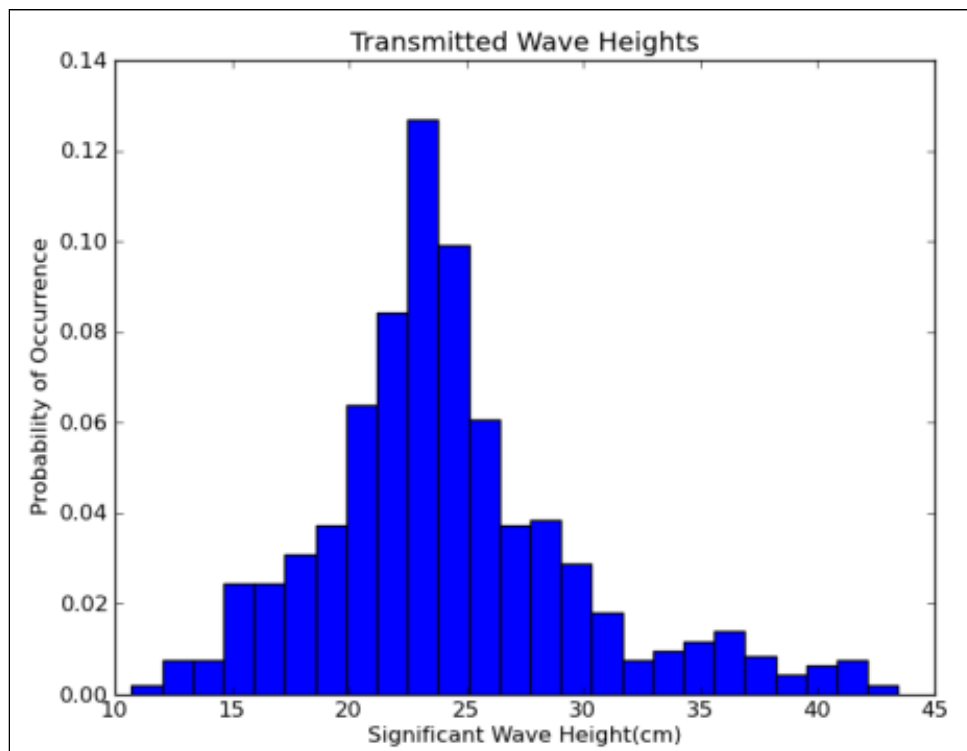
5.3.2.3   Evaluation of terminal groin

Two configurations adding groins to the breakwater field were modeled
with CMS. The configuration shown in Figure 45 has a groin added approxi-
mately 600 ft to the southwest of the breakwater field, adjacent to Mitchell's
Cut. The groin tip extends the same distance from shore as the breakwaters.
The yellow in the figure shows erosion, and blue shows accretion. Figure 46
shows the CMS result with the same groin placed 300 ft southwest of the
breakwaters. This configuration allows the groin to essentially act as a
breakwater, and the results show no erosion between the breakwater and
the groin.

Both configurations show increased erosion adjacent to Mitchell's Cut, due
to the groin interfering with longshore current flow and corresponding
sediment transport in the vicinity of the inlet. Due the increased erosion
near Mitchell's Cut and trapping within the breakwater field, construction of
the groin is not recommended for the demonstration project. The addition
of a terminal groin should be re-evaluated after the demonstration's
monitoring data have been analyzed.

Figure 45. Groin added 600 ft downdrift (southwest) from breakwater field.
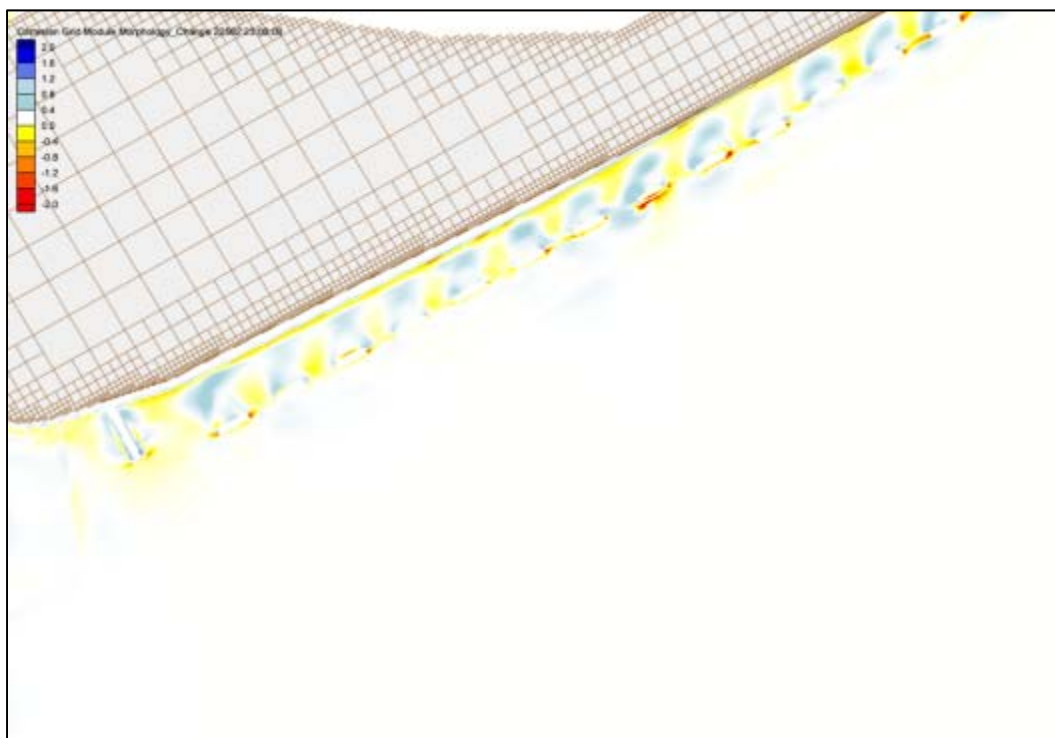
Figure 46. Groin located 300 ft southwest from breakwaters.
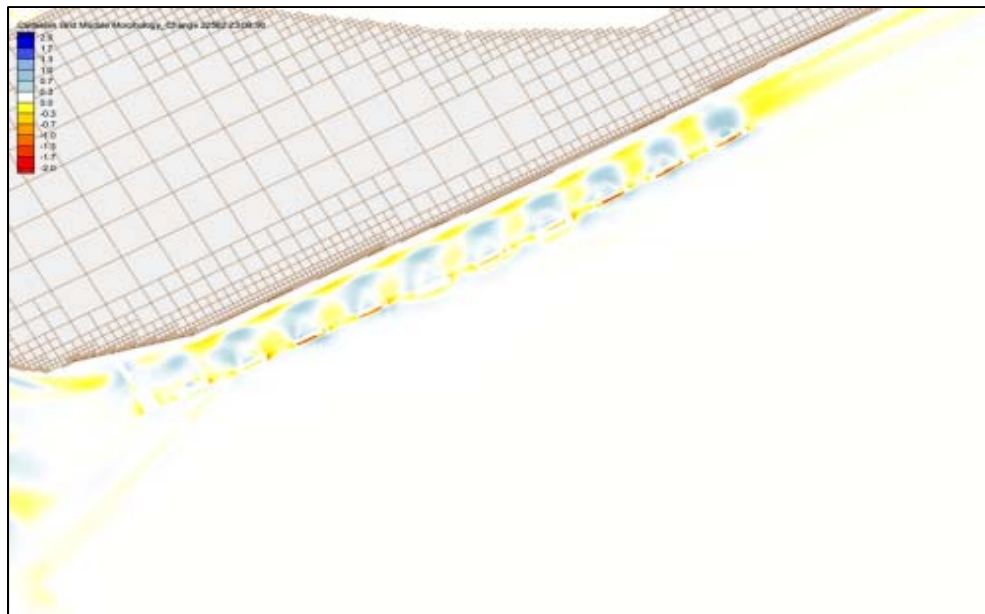
5.3.2.4   Currents

Modeling the nearshore environment of the breakwaters with CMS helped identify conditions that violate or stretch the assumptions inherent in the shoreline change model. For example, currents might introduce a bias in the shoreline model that is not included in its calibration. Figure 47 shows CMS with a longshore current resulting from incident waves under normal conditions.

Figure 47. Typical currents in the breakwater field.
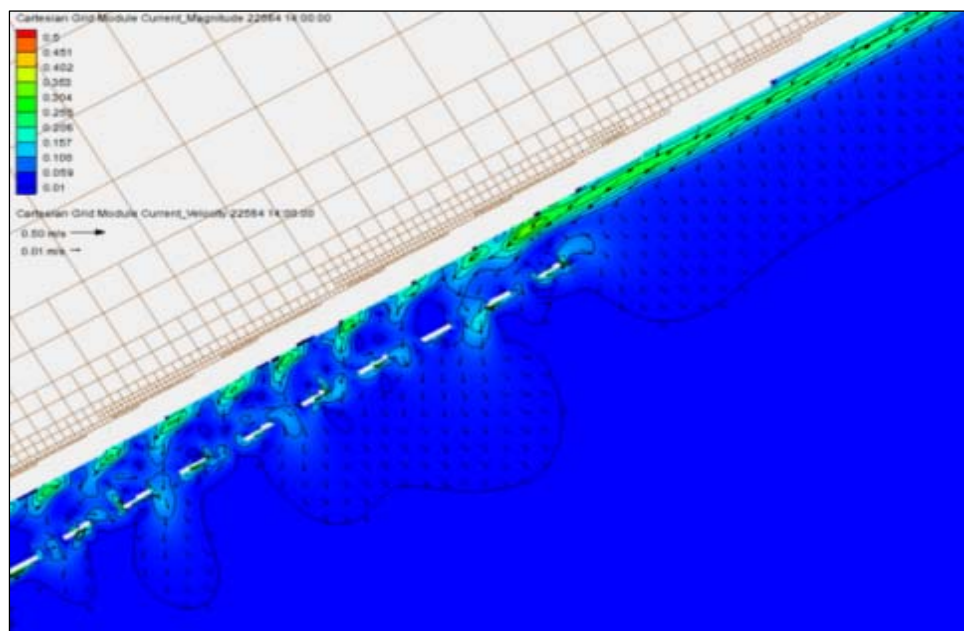
A result of the CMS model is to use the formula by Komar and Inman (1970) that predicts the longshore current velocity at the midpoint of the surfzone:

$$V_l = 1.17 \sqrt{gH_b} \, \sin\theta_b \cos\theta_b \qquad (5)$$

where

H_b = Wave breaker height;
g = acceleration due to gravity;
θ_b = breaking wave angle relative to the shoreline;
V_l = magnitude of longshore current.

In Figure 47, the wave angle of incidence is 5 deg, and the breaking wave height is 0.5 m. The result from applying the equation matches the observed longshore current of 0.2 m/sec. These results are not valid when the waves are not the primary factor influencing the longshore current. (For example, high winds parallel to the shoreline will affect the speed of the longshore current).

Currents observed outside the surf zone are considerably less than near-shore currents, which is consistent with shoreline model assumptions. The CMS results also show that longshore currents impinging on breakwaters or constrictions caused by significant (tens of feet) variations in the breakwater to shoreline distance can intensify erosion.

CMS includes wave setup and cross-shore currents that might occur due to the difference in water levels between the shoreline and offshore. Since the breakwaters moderate the waves and corresponding water levels along the shoreline, cross-shore flow occurs as shown in Figure 47, where the blues represent slow current speed and green represents currents approximately 0.2 m/sec (0.6 ft/sec).

5.4 Recommended breakwater design parameters

As stated, a Holly Beach analog is recommended for further study with specific refinements based on numerical model analysis. A combination of analysis techniques resulted in the following general design parameters:

- 6 ft depth;
- 4 ft height;

- 220 ft long segments;
- 330 ft long gaps;
- 350-400 ft distance from shore.

5.5 Discussion of breakwater system feasibility

Because of the low ratio of net transport to gross transport and the presence of cohesive sediments, breakwaters appear to be the most cost-effective and practical solution to reduce erosion and maintain a beach habitat at Sargent Beach (Thomas and Dunkin 2012). Unfortunately, the risk of adjacent impacts to the littoral system is high compared to beach nourishment. Breakwater design is sensitive to understanding physical processes, a noted problem at Sargent Beach. Although technical analysis indicates break-waters are feasible, a demonstration project that can be adapted later if needed (e.g., adding stone to lengthen breakwaters and decrease gap widths, or removing stone to increase wave transmission and overtopping) is the preferred method of proving low-cost feasibility with minimal impact to the environment.

5.5.1 Environmental considerations

Environmental considerations for breakwaters include downdrift erosion, beach-shape modification, and increased turbidity during construction. Breakwaters could restrict access for nesting sea turtles, and currents generated in gaps could inhibit turtles from coming onshore, although USACE (1993) reports that turtles avoid Sargent Beach. Environmental impacts should be evaluated while monitoring the demonstration project.

5.5.2 Habitat protected

The proposed demonstration project would protect the region behind the breakwaters shown in Figure D3.

5.5.3 Storm damage vulnerability

The breakwater system would curb erosion within the target area, increasing storm damage protection. A minimal beach fill is recommended with initial construction. Replacement of eroded sand after a storm will likely be required, as well as the repair of natural beach damage. Because the structures will be in shallow water, recommended stone size is much smaller than typical on the Texas coast. They will likely require repair after a major tropical storm or hurricane.

For an engineered project to be eligible for federal assistance after a catastrophic event, the project must have a maintenance plan and the sponsor must have the means and intent to maintain the project (described in Chapter 4.) The plan should include the following: project description, maintenance plan and actions, and project funding.

5.5.4 Constructability

The project dimensions are based on the assumption that the breakwaters will be constructed using land-based techniques. A sand bridge to the construction depth would facilitate equipment access; the limited nourishment included in the design is intended to facilitate this process. Geotechnical stability of the soils in the nearshore is uncertain and might impact breakwater placement or design options. Geotechnical investigations and analyses are required before final design.

5.5.5 Regional considerations and enhancement

The breakwater project is designed to trap about 50 % of the net longshore transport leeward of the breakwaters. Because the net transport rates are relatively low and a small number of breakwaters is recommended for Phase 1 construction, the downdrift effects are expected to be small, not more than 40,000 yd^3/yr. The volume trapped by the breakwaters will not reach the land south of Mitchell's Cut, so mitigation material might be necessary on its southern side. It could be placed as a dune feature or beach fill. Dunes would naturally restore the beach during and after storms and help minimize overwash into East Matagorda Bay. The volume necessary for mitigation during Phase 1 equals about 7 yd^3 of sediment per ft per yr over a 1-mile segment just south of Mitchell's Cut. A larger amount of sediment would be trapped by a full scale breakwater project.

5.6 Preliminary breakwater design

The recommended project at Sargent Beach consists of 10 segmented shore-parallel breakwaters and a minimal beach fill to pre-fill behind the break-waters. A groin to the east of Mitchell's Cut is not recommended. This section discusses preliminary design of the breakwaters, limited to the scope of the recommended Phase 1 project that includes 10 breakwater segments.

5.6.1 Armor size

Due to the hydraulic instability of rubble-mound structures, it is necessary to use empirical formulas to calculate the stone weight required to withstand design waves. The Hudson and van der Meer equations were applied to determine the median weight of an individual armor unit in the primary cover layer, W_{50}, presented in greater detail in Chapter 4. The presentation is shortened in this chapter for brevity.

To determine waves acting on the structure, they were transformed to a depth just offshore of the structures using CMS-Wave. Nearshore waves were calculated as a function of water level from -1.9 to 7.7 ft (NAVD88) and the 50-yr return period for offshore waves. Guidance from Battjes and Groenendijk (2000) was used to determine the statistical wave heights. This method was applied to determine the 10 % exceedance wave height, $H_{10\%}$, for input into the Hudson stone stability equation and significant wave height, H_s, for the van der Meer stone stability equation.

The unit weight of the stone was assumed to be 155 lb/cf with side slopes of 3H:1V. A stability coefficient of 2.2 was selected based on guidance from USACE (2002). Graded riprap armor stone is recommended, similar to the Holly Beach breakwaters. A graded riprap gradation from 2,000 lb to 6,000 lb is recommended for the breakwaters, a commonly applied gradation in Galveston District projects. Considering reductions in structure elevation for overtopping and submergence, this gradation should be adequate for a single 50-yr return period storm. It is likely that the structure will require repairs during its design life; however, because the structure is intended to be adaptive, smaller stone is recommended to reduce initial cost and future cost of adaptive modifications. Stone size should be evaluated in more detail during design phases before construction.

5.6.2 Suggested breakwater section

Typical breakwater plans, cross sections, and profile are shown in Appendix D. Justification for selected parameters is provided in this section.

5.6.2.1 Crest width

The minimum crest width is a function of the stone size, $Dn_{50} = 3$ ft, for breakwaters. Crest width is specified at 10 ft, rounded up from the recom-

mended minimum width of three times Dn_{50}. Minimum armor layer thickness below the crest is specified as 6 ft (2 times Dn_{50}).

5.6.2.2 Scour protection

Detailed scour calculations have not been conducted in this preliminary design. No additional scour stone is included. Scour design should be conducted during final design. Marine mattresses should be part of a cost vs. constructability tradeoff.

5.6.2.3 Side slopes

All side slopes were set at 3H:1V. Steeper side slopes are allowable and might reduce stone cost, but will result in larger required armor stone. The flatter side slope was chosen to minimize armor size and to reduce overall bid costs by making structures easier to build.

5.6.2.4 Typical section

The graded riprap armor layer should be directly on a bedding layer with median weight ranging from about 2 lb to 25 lb. A 1.2-ft thick bedding layer is recommended. Note that a scour hole might form during placement, requiring more fill than shown in the neat line approximations. Typical breakwater plan and cross sections are shown in Appendix D (Figures D4 and D5). Figure 48 plots a simplified breakwater cross section.

Figure 48. Graded riprap breakwater cross section (details shown in Appendix D).
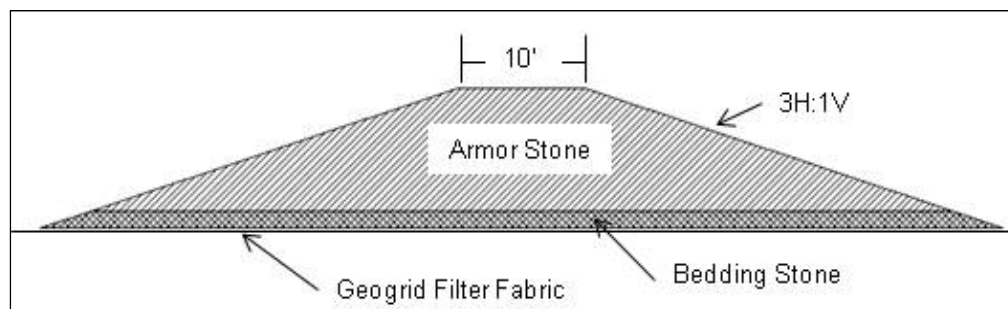
5.6.3 Beach fill design

A limited beach fill is recommended for Sargent Beach. It serves two main functions:

- It pre-fills the template along the shore to be protected, reducing the amount of natural sediment trapped.
- It provides sand for construction operations.

The minimum beach fill width is governed by the amount of "erosion," or negative response of the shoreline landward of the existing shoreline due to the formation of salients. This beach fill does not include fill potentially needed to mitigate for the project, discussed in the following section.

As discussed earlier, the structure to shoreline distance is a main factor in determining the effectiveness of the breakwaters. Once the breakwaters are placed, beach fill is the only practical means to vary the shoreline to break-water distance. Expected shoreline erosion without the project averages about 12 ft/yr. With breakwaters in place, the average shoreline change is about 3 ft of accretion per yr. Some short stretches of the shore could experience up to 15 ft of erosion per yr, but this is a result of redistribution of sand to form salients behind the breakwaters.

A preliminary design based on future conditions is given to facilitate the project lifecycle, resulting in about 5 yd^3 per linear ft. A detailed analysis was conducted to determine the volume of fill necessary to protect the shoreline behind the breakwaters. Over the first 5 yr after 15 breakwaters are placed a distance of 350 ft offshore, the shoreline accretes 15.5 ft on average. The number includes erosion due to the formation of salients. Since the overall shoreline trend after 5 yr is to advance, only the locations of erosion behind the breakwaters were studied. The average amount of erosion in those areas was -36.8 ft.

To determine the volume loss associated with shoreline erosion, the shoreline erosion was multiplied by the distance of alongshore erosion areas and the sum of the berm height and depth of closure. Although the depth of closure at Sargent Beach is close to 20 ft, the depth of sand closure is assumed to be 4 ft due to limited transport behind the breakwaters (Mann et al. 2004). The total volume of erosion in those sections over the 5-yr period is about 14,200 yd^3. Based on the total volume of erosion, the volume of sand placed per linear foot ranges from 2 to 10, depending on where the fill is placed. If the entire beach from Breakwater 1 to Breakwater 15 was filled with 14,200 yd^3, this results in a volume of 2 yd^3 per linear ft. However, if sand was only placed in erosion spots, the resulting volume is 10 yd^3 per linear ft. The volume necessary will change based on the analysis

and where the fill is to be placed. Through this analysis and engineering judgment, a volume of 5 yd^3 per linear ft was chosen.

Figure 49 shows the typical beach fill construction template. A more detailed figure is included in Appendix D. The construction template includes the entire volume of fill that will be distributed over the beach profile by natural processes; therefore, the added berm width for the construction template is much greater than the design profile.

Figure 49. Beach fill design at Sargent Beach.
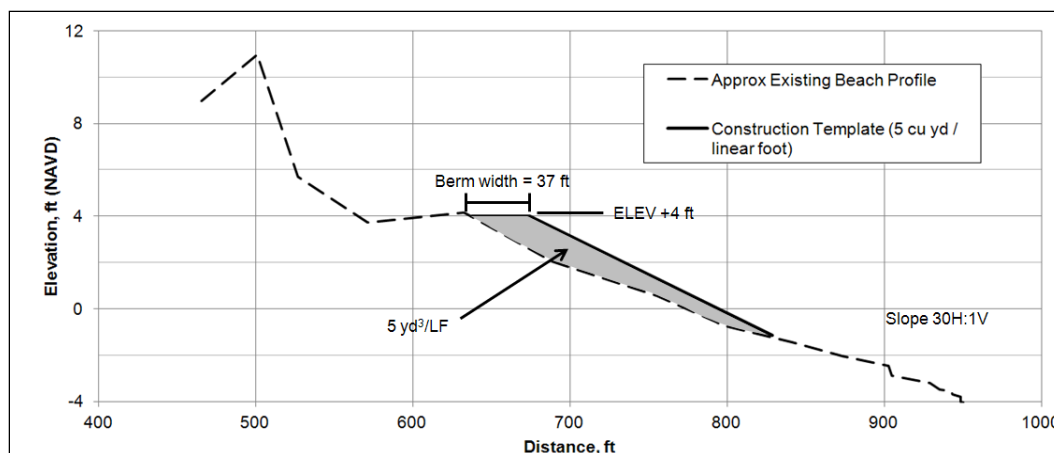
5.7   Preliminary opinion of probable construction cost

Table 12 was prepared based on the design presented above, discussion with SWG cost engineers, and previous USACE projects. This preliminary opinion is intended to be used to help Port of Bay City Authority determine how to proceed with funding and constructing a shoreline stabilization project. The opinion should be updated during final design when more accurate estimates of material costs and quantities are available. A per-breakwater cost is given to assist the funding of additional construction phases.

Table 12. Sargent Beach breakwater preliminary opinion of cost.

ITEM	QUANTITY	UNIT	UNIT PRICE		EXTENSION	
Mobilization / Demobilization	1	LS	$	400,000	$	400,000
Topo / Hydro Acceptance Surveys	1	LS	$	75,000	$	75,000
Hazard (Magnetometer) Survey	1	LS	$	30,000	$	30,000
Site Preparation and Restoration	1	LS	$	100,000	$	100,000
Breakwater Construction	10	Each	$	710,155	$	7,101,550
			CONTINGENCIES (30%):		$	2,311,965
				SUBTOTAL:	$	10,018,600
BREAKWATER UNIT COST						
Armor Stone	5,210	Ton	$/Ton	100	$	521,000
Blanket Stone	1,520	Ton	$/Ton	70	$	106,400
Geogrid Composite	2,390	SY	$/SY	15	$	35,850
Beach Fill	2,650	CY	$/CY	18	$	46,905
	SUBTOTAL PER BREAKWATER:				$	710,155
				TOTAL:	$	10,018,600

Notes
1. Cost of stone based on estimate from SWG planning guidance (Regner 2012).
2. Quantities and unit prices are based on approximate in place dimensions.
3. This Opinion of Construction Cost is based on data available at the date of publication and is not necessarily all inclusive of cost items. Actual construction costs may vary based on changes in market conditions.
4. Cost of beach fill based on Coastal Tech (2010) truck haul estimate. Cost of sand provided by Mike Griffith.
5. Cost of beach fill for breakwaters includes cost of sand, transportation, and installation.
6. Volume of beach fill based on fill volume for Phase 1 project length, averaged over 10 breakwater segments.

6   Project Implementation

The recommended designs on both Matagorda Peninsula and Sargent Beach are based on uncertain data and analyses. Therefore, an adaptive approach is recommended for project implementation. This chapter summarizes the sources of uncertainty and risk, recommends a plan for implementing structural solutions, and suggests a general monitoring approach to enable the decision-making process.

6.1   Matagorda Peninsula groin system

On Matagorda Peninsula, the new east jetty at MCR constructed in 2010 has caused a change in the equilibrium conditions that has not yet been monitored or documented. Design documents for the new jetty (Kraus et al. 2008) predict that 200,000 yd^3/yr of mechanical bypassing from the north to south sides of the jetty will be required, but funding for that effort is not guaranteed. In addition, there is inherent risk associated with predicting beach response to structures based on simplified models forced with historical data. This uncertainty in how beaches east of MCR will respond to the new jetty increases the uncertainty associated with beach response to the new groin system.

6.1.1   Implementation plan

The proposed plan documented in Chapter 4 and Appendix D calls for construction of three new groins and an optional beach fill. Since performance of the shoreline stabilization project is heavily dependent on natural bypassing at MCR, no structures should be constructed until sufficient data has been gathered and analyzed to determine the influence of the new east jetty on adjacent beaches and inlet dynamics.

A monitoring project, described in the following subsection, should be undertaken to determine the rate of natural bypassing at MCR, rate of sediment trapping and shoreline change to the northeast, and rate of sediment lost and shoreline change to the southwest. Model configurations were tested and presented in Chapter 4 and Appendix A to characterize the potential influence of various bypassing rates. These results should be applied after monitoring data have been analyzed to determine the final optimum groin and beach configuration to meet the projects' stated goals.

The volume of fill placed is dependent on the desired added dry beach width. Determining the amount of fill to place should be based on economic needs and predicted performance. Beach maintenance would be required to prevent dune advance with shoreline position, if that remains a project goal.

After the pre-construction monitoring data have been analyzed and the final configuration decided upon, the system should be constructed starting with the downdrift (southwestern) groin, then completed in order towards the northeast. Beach fill, if opted for, should be placed as the groins are completed.

After construction, monitoring should be conducted to maintain FEMA eligibility, inform structure maintenance, and enable system tuning. Although the structures are designed to be stationary, it is possible to raise, lengthen, or reduce porosity if monitoring data suggest insufficient sediment is trapped. The converse would also be possible if data indicated it necessary.

6.1.2 Monitoring plan

Monitoring is required before construction to determine the optimum configuration and after construction to evaluate structure maintenance needs and system performance.

6.1.2.1 Pre-construction monitoring plan

Monitoring should be conducted until MCR has reached a new state of dynamic equilibrium before construction begins e.g., 5 or more years. Building a groin system without understanding inlet processes could lead to larger than predicted maintenance or mitigation requirements. Goals of monitoring and recommended actions are summarized below.

Pre-construction monitoring goals:

- Quantify natural bypassing at MCR;
- Quantify shoreline change on both sides of MCR and over the project area;
- Recommended monitoring actions:
  - Semi-annual condition surveys at the MCR entrance, extended to capture any developing ebb shoal;
  - Semi-annual georeferenced vertical aerial photography.

Recommended analyses:

* Aerial photographs should be analyzed to determine shoreline change;
* Surveys should be analyzed and coupled with the shoreline change analysis and any dredging and placement activities to determine the bypassing rate.

6.1.2.2 Post-construction monitoring plan

Post-construction monitoring should be conducted in perpetuity to maintain eligibility for federal assistance after storms and ensure desired system performance. Goals of monitoring and recommended actions are summarized below.

Post-construction monitoring goals:

* Quantify performance of the groin system;
* Provide data for structure and beach maintenance.

Recommended monitoring actions:

* Annual beach profile surveys within and adjacent to the groin field;
* Quarterly and post storm visual inspection of structures and beaches;
* Annual georeferenced vertical aerial photography.

Recommended analyses:

* Limited analysis and documentation of collected data will enable FEMA eligibility and provide data needed for system maintenance.

6.2 Sargent Beach Phase 1

Sargent Beach is unique on the Texas coast, experiencing some of the highest erosion rates recorded. Beach sediments include silts and clays, not just sand. There is an 8-mile revetment protecting the GIWW in imminent danger of being exposed to continuous wave action. Net transport is small relative to gross transport. The recommended plan to reduce erosion and protect habitat includes construction of segmented shore-parallel breakwaters, the most sensitive of coastal protection structures. Combined, these factors make it impossible to reduce risk to an acceptable level to

move forward with full-scale construction. Therefore, a fully adaptive approach to project implementation is recommended.

The central concept to the adaptive approach is that greater uncertainty exists than can be accounted for through present scientific understanding. By building one or more smaller demonstration projects, then monitoring performance and revising structure design, it will be possible to evaluate performance and potential negative outcomes in a manageable way with less risk and dependence on science not yet understood. The recommended implementation and monitoring plan are provided in following subsections. The general adaptive process recommended is:

- Define goals or objectives;
- Develop an understanding of the environment and create/modify a plan based on the best available science and data;
- Take action;
- Monitor the project;
- Analyze monitoring data;
- Start over at step 1, reducing risk through each iteration.

6.2.1 Implementation plan

This report documents the recommended first phase of construction in the adaptive plan to protect beaches and habitat at Sargent Beach. The proposed Phase 1 plan presented in Chapter 5 and Appendix D calls for construction of 10 new shore parallel segmented breakwaters with a limited beach fill. The recommended initial breakwater field extends from just northeast of Mitchell's Cut, heading northward, in a location where unexpected effects should not adversely affect the revetment or inlet.

Identifying project success criteria is critical to successful adaptive approaches. Success criteria for Phase 1 are:

6.2.1.1 Structural success criteria

- No significant scouring or settlement;
- No significant displacement of armor stone;
- Structure crest height maintained within given tolerances;
- Wave overtopping as expected.

6.2.1.2   Sediment success criteria

- Trapping 50 % of the incident longshore transport under normal conditions;
- Reduced/no net shoreline recession behind the breakwater field;
- Downdrift placement has minimal/no impact on Mitchell's Cut;
- Mitchell's Cut remains stable with breakwaters placed;
- Structure placement does not cause excessive trapping end effects or tombolo formation over the project design lifetime;
- The predicted amount of mitigation material is approximately equal to the amount of material trapped by the breakwaters (predicted to be less than 40,000 yd^3/yr).

The breakwaters are intended to be constructed from the downdrift direction of longshore transport towards the updrift (from southwest to northeast). The limited beach fill is intended to pre-fill the template to reduce initial sediment trapping. The fill could be used to help enable land-based construction, although means and methods of construction are not intended to be specified in this document. When completed, the fill should be graded to approximate shoreline orientation during or after construction.

After construction of Phase 1, the system should be monitored for at least a year, or until performance and impacts are conclusively measured. The recommended monitoring plan is based on the success criteria noted above and described in Section 6.2.2. Then lessons learned should be applied to design of subsequent phases and modification of the Phase 1 breakwaters. Some potential cases that might result in design modification are presented in Section 6.2.1.1 to illustrate potential adaptations. After collecting and analyzing data, the project team should discuss the results, determine how well the preliminary analyses represent observed conditions, and consider preferred alternatives for future actions.

Iterations through the adaptive process should result in a better design that more completely meets the specified success criteria. Many different approaches to protect all of Sargent Beach were considered through application of numerical models documented in Chapter 5 and Appendix B, as well as in the initial feasibility study (Thomas and Dunkin 2012). Discussion of potential future phases of breakwater construction is located in Appendix B. The preferred structural solution will be based on an analysis of Phase 1 monitoring data (10 breakwaters).

6.2.1.3   Potential cases for corrective actions

This subsection describes some likely cases requiring modifications to the proposed design and action that might be taken. These cases are provided as an example and are not inclusive of all design adaptations that could result from monitoring data.

Case: Excessive salient or tombolo formation

Remove the top layer of rock from the breakwaters to increase the rate of overtopping – energy transmitted through the breakwaters to the beach. Or, remove some stone from the ends of each breakwater, increasing gap width.

Case: Greater erosion than expected for a given storm climate.

Possible actions in this case include a beach fill to decrease shoreline-to-structure distance. Also, the breakwater gap size could be decreased, or the structure height increased, by adding stone. If funding is available, then sand could be stockpiled in dune features, stabilized with vegetation and fencing, and then moved quickly onto the beach when needed while providing additional storm protection.

Case: Mitchell's Cut stability

If the mitigation material is placed too close to the inlet, then future placements should be placed further downdrift. Groins might be necessary to prevent excessive influx of sediment to the inlet if the sediment cannot be placed far enough away.

6.2.2   Monitoring plan

Data collection is integral to evaluating the success of Phase 1 and informing design of a successive larger scale project. Monitoring should continue for at least a year or until performance and impacts are conclusively measured. Weather during the monitoring period will likely control the length of time monitoring is required. Goals of monitoring and recommended actions are summarized below.

Monitoring goals:

- Quantify shoreline change in and adjacent to the project area;

- Quantify volume trapped and bypassed at the breakwater field and the volume lost on adjacent beaches;
- Quantify stability of Mitchell's Cut;
- Quantify wave transmission through breakwater field.

Recommended monitoring actions:

- Monthly surveys of the beach and Mitchell's Cut during the first three months after construction. Surveys should be collected biannually and after energetic storms;
- Quarterly surveys and inspection of segmented breakwaters;
- Continuous water-level monitoring in the GIWW at FM 457;
- Quarterly georeferenced vertical aerial photography;
- Deploy wave gauges seaward and landward of the breakwaters.

Recommended analyses:

- Aerial photographs should be analyzed to determine shoreline change;
- Surveys should be analyzed and coupled with the shoreline change analysis to determine volume trapped and lost on beaches behind and adjacent to the breakwater field;
- Water level data should be used to help evaluate project performance and to determine how well actual conditions relate to modeled conditions;
- Breakwater surveys and inspections will enable analysis of structural performance and inform design adaptations;
- Analysis of Mitchell's Cut morphology should be conducted to ensure stability;

Analysis of wave data will enable direct calculation of wave transmission coefficients to better refine crest elevation, gap width, and distance offshore.

7 Summary and Conclusions

This report documents preliminary design of a groin system on Matagorda Peninsula and segmented breakwaters at Sargent Beach. A summary of results and recommendations at each site are presented in this chapter.

7.1 Matagorda Peninsula

The goal on Matagorda Peninsula is to halt erosion and increase beach width over the target area to accomplish the final project goals:

- Groin field to stabilize the beach from the north access road to 3 Mile Cut;
- Increase the dry beach width by 200 ft over this area;
- No impact to shoreline change rates at 3 Mile Cut;
- Bypassing plants at MCR will not be included.

Numerical modeling was conducted to help design the groin system and to quantify system performance and impacts to adjacent beaches. The recommended groin design is shown in Appendix D and summarized below:

- Three groins

    o 980 ft total crest length each
    o 8-ton armor stone
    o 1,600-ft spacing

- Optional beach fill

    o None, 50-, 100-, and 200-ft added berm widths
    o Selected initial beach fill should be based on project goals and pre-construction monitoring

- Preliminary opinion of construction cost

    o No fill: $12,348,600
    o 50 ft fill: $15,675,800
    o 100 ft fill: $19,003,000
    o 200 ft fill: $25,657,400

Analysis for design of the groin system highlighted one major limitation that should be considered before construction:

- Bypassing at MCR controls groin system performance

  - Since the new east jetty was constructed in 2010, natural bypassing is unknown and the system is still coming to a new equilibrium;
  - Numerical models of the new jetty (Kraus et al. 2008) suggested that as much as 200,000 yd^3/yr would need to be mechanically bypassed (dredged) to maintain shoreline position;
  - Monitoring of natural bypassing should be conducted before final design and construction.

Implementation and monitoring plans are included in Chapter 6 to help guide final design, construction, and monitoring. The following recommendations are included:

- Monitor the region before final design of a new groin field to quantify natural bypassing at MCR;
- Revise the groin system design based on pre-construction monitoring to help refine performance associated with MCR bypassing;
- After construction, monitor the project to maintain eligibility for federal assistance.

7.2 Sargent Beach

The goal on Sargent Beach is to halt erosion over the target area shown in Figure 1. Due to the risk in predicting project performance associated with cohesive sediments and design of breakwaters, a demonstration project (Phase 1) is recommended. Numerical modeling, empirical analysis, and evaluation of nearby and similar projects was conducted to help design the breakwater system and to quantify system performance and impacts to adjacent beaches. The recommended Phase 1 design is shown in Appendix D and summarized below:

- Ten breakwater segments

  - 220- ft segment crest length
  - 4-ft NAVD crest elevation
  - 330- ft gap width
  - 1-3 ton armor stone
  - Approximately 350 ft offshore from present shoreline

- 5 yd^3/lf beach fill
- Preliminary opinion of construction cost

- o $10,018,600 total for 10 segments
- Less than 40,000 yd³/yr erosion predicted southwest of Mitchell's Cut resulting from breakwater construction

Cohesive sediments are not well understood or represented in the numerical models applied. Breakwater design is sensitive to environmental forcing and selection of many different design parameters including crest elevation, gap width, segment length, and distance from shore. Therefore, an adaptive approach to project implementation is proposed. Highlights of the recommended approach are listed below:

- Construct 10 segment breakwater demonstration project (Phase 1 construction)

  - o   Design based on empirical formula, nearby/similar projects, numerical model analysis.

- After Phase 1 construction, monitor to determine:

  - o   Shoreline change in and adjacent to the project area;
  - o   Volume trapped and bypassed at the breakwater field and the volume lost on adjacent beaches;
  - o   Stability of Mitchell's Cut;
  - o   Wave transmission through breakwater field.

- Evaluate monitoring data:

  - o   Determine how well design conditions match observations;
  - o   Quantify observed adjacent impacts.

- Modify structure design to plan the next phase of construction.

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Appendix A: Numerical Modeling with GenCade

Calibration and validation

Summary of model parameters

The table below provides a summary of the model input parameters for calibrations. The majority of these parameters did not change from the Phase 1 report calibration.

Table A1. Summary of model input parameters for calibration.

Parameter	Value
Start Date	1/1/1995 0:00
End Date	12/30/1999 0:00
Time Step	0.1
Recording Time Step	168
Effective Grain Size, mm	0.2
Average Berm Height, ft	4.0
Depth of Closure, ft	19.7
Left Lateral Boundary Condition, moving (ft per sim period)	217
Right Later Boundary Condition, moving (ft per sim period)	92

Model calibration

In this application, numerous iterations of GenCade were evaluated by comparing calculated net and gross transport rates and shoreline change to available measurements or estimates. Table A2 lists the parameter values found to best represent the observed data through the calibration process. Goodness of fit was based on root mean square error between the calculated shoreline after 5 yr from 1995 to 2000 and the observed 2000 shoreline position.

In Table A2, K1 and K2 are sand transport rate coefficients, and the values selected are similar to those selected at other locations on the Texas coast (King 2007). K1 and K2 used here are lower than the values used in the set up for Phase 1, but the values meet the guidelines for selecting K1 and K2.

Table A2. Model parameters selected through calibration.

Parameter	Value
K1	0.2
K2	0.1
Height Amplification Factor	1
Angle Amplification Factor	1
Angle Offset	5.5°
Number of cells in offshore contour smoothing window	11

The Height and Angle Amplification Factors are set equal to 1, indicating no increase or reduction of the measured values. An angle offset is applied to force net transport to the west. This factor is commonly applied at locations with coarse wave data or where there is very low net transport, like Sargent Beach. Without this factor, the model calculates low net transport to the east of Sargent Beach, contrary to common knowledge and observed processes. Finally, the number of cells in the offshore contour smoothing window specifies the length over which the shoreline is smoothed to develop a representative offshore contour for wave refraction calculations. In Phase 1, the ISMOOTH value (number of cells in offshore contour smoothing window) was 50. This value was decreased to 11 to improve the model functionality when breakwaters are included in the set up.

Comparison between calculated net and gross longshore transport rates to those estimated by others are shown in Figures A1 and A2, respectively. Figure A3 compares the calculated and observed shoreline change rates from 1995 to 2000.

Table A3 shows two goodness-of-fit statistics applied to evaluate the GenCade model; root mean square (RMS) error and Brier skill score (BSS).

Both goodness of fit statistics in Table A3 show that the model performs well between SBR and west of Mitchell's Cut, after calibration was complete. Initially, the GenCade model under-calculated shoreline loss by over 1,000,000 yd³/yr between SBR and MCR. After exhaustive variation of model parameters, the only method to reproduce shoreline recession in the region was to extract an additional 1,314,000 yd³/yr (150 yd³/hr) from the Sargent Beach and West of Mitchell's Cut cells. Recall from the sediment budget discussion that 878,575 yd³/yr of sediment remains as a residual loss in the budget in the region, a conclusion so supported by Seelig and Sorensen (1973).

Figure A1. Net transport, model results vs. published values for the calibration case.
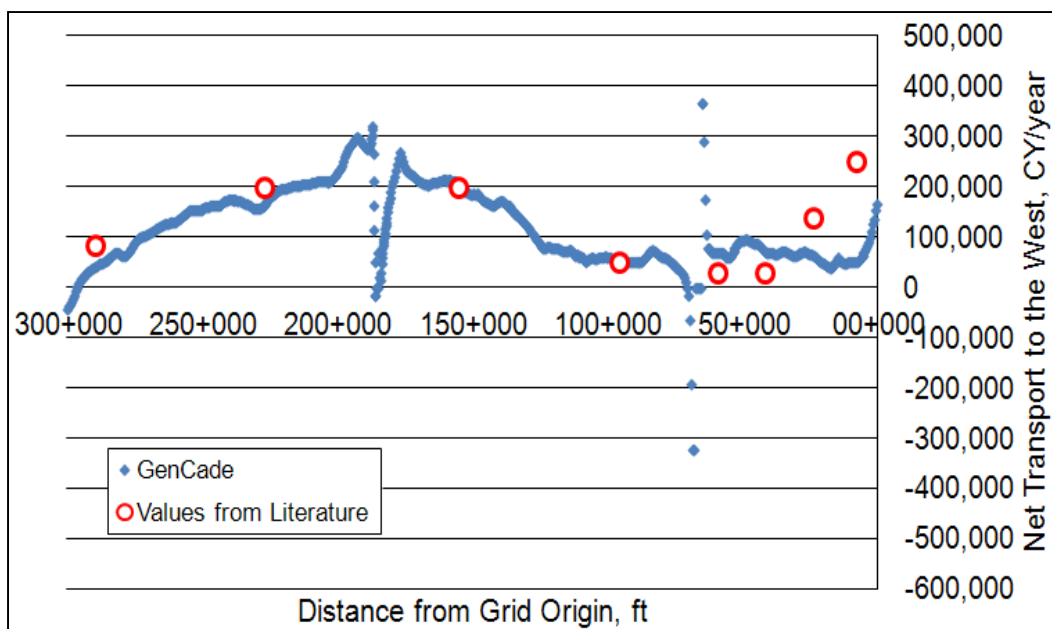

Figure A2. Gross transport, model results vs. published values for the calibration case.
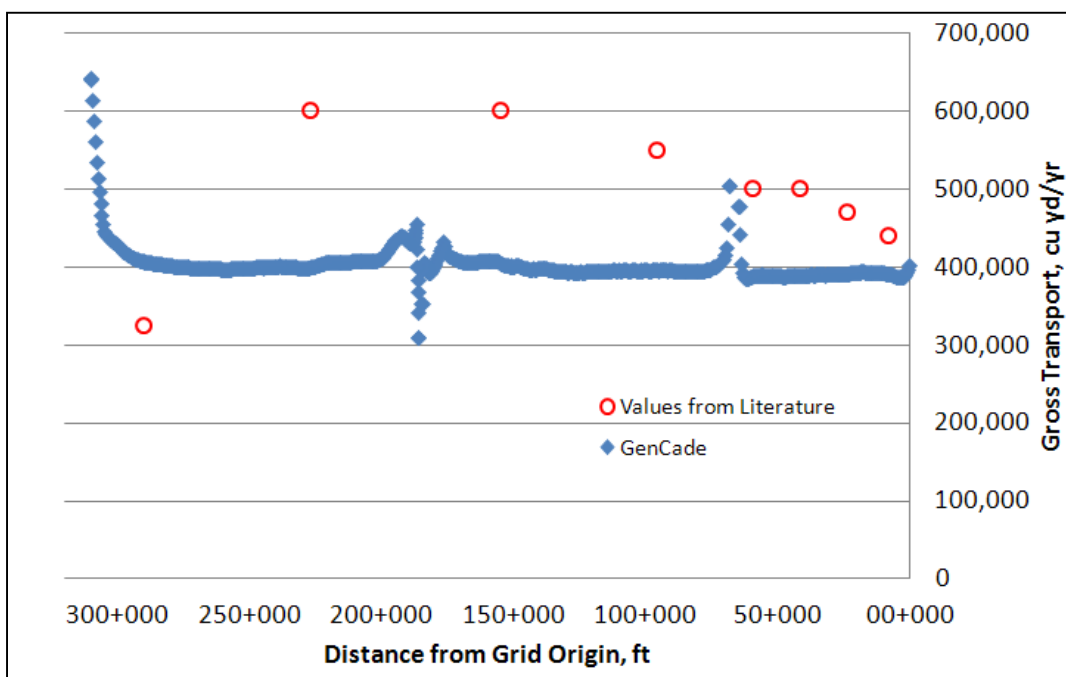
Figure A3. Shoreline change, model results vs. published values for the calibration case.
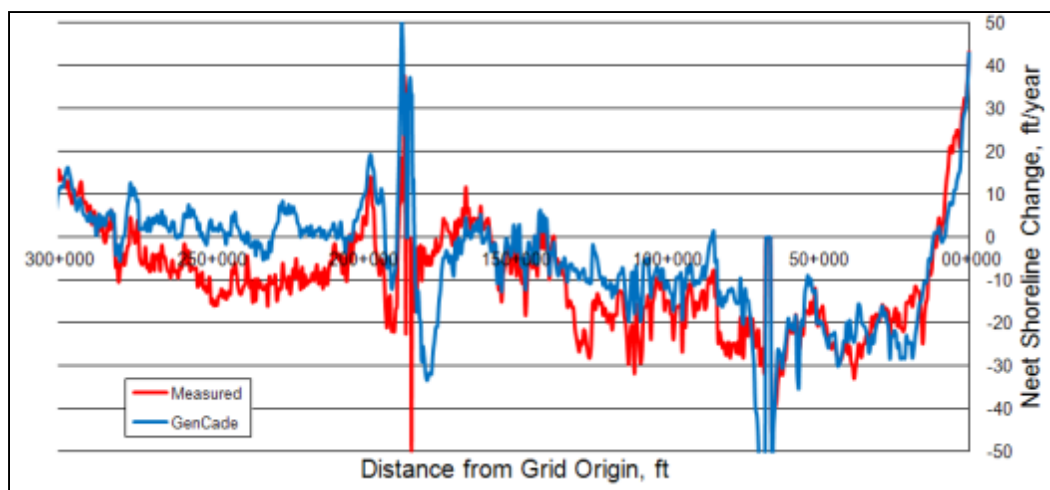

Table A3. Shoreline change modeling statistics.

Cell	Average Shoreline Change, ft/yr		RMS Error, ft/yr	Brier Skill Score
	Measured	Modeled		
SBR to Cedar Lakes	10.4	7.4	7.4	0.87
West of Cedar Lakes	-17.2	-22.3	6.5	0.86
Sargent: East of FM 457	-23.9	-22.3	4.5	0.97
Sargent: West of FM 457	-26.1	-24.5	4.7	0.97
West of Mitchell's Cut	-18.6	-13.6	8.6	0.80
East of MCR	-5.7	-6.2	10.4	-0.10
MCR to MSC: North	-7.2	4.2	12.5	-0.57
MCR to MSC: South	6.0	6.7	6.9	0.54

The requirement to introduce volume loss into the model in this manner indicates that a vital process is not included in the model. It is likely that the missing process is a combination of cross-shore losses, loss of fine grained sediments, and trapping at the inlets as well as error typical with this class of model. Unfortunately, the GenCade model has not progressed to the point where cross-shore processes are included, so extracting the 1,314,000 yd3/yr is the only way to calibrate the model. Since analysis of shoreline change at Sargent Beach includes so much uncertainty associated with the cohesive sediments, shoreline response might be different if sand were placed over the cohesive sediments. Various alternatives to be evaluated in this study include beach nourishment. Therefore it is possible that the model will over-calculate erosion rates for those alternatives (i.e., model calculations will be conservative).

1991-1995 setup

After the model was calibrated for the 1995 to 2000 time period, the model was run for the 1991 to 1995 time period. The 1991 shoreline does not extend the entire GenCade domain, so the 1990 shoreline was taken as the initial shoreline for approximately the northernmost 15,000 ft.

The model domain is identical to the 1995-2000 simulation. The inlet shoal volumes, sediment bypassing, and model parameters are also identical to the 1995-2000 simulation. Data were taken from the same WIS hindcast stations as the 1995 to 2000 simulation.

Comparison between the calculated net longshore transport rates to those estimated are shown in Figure A4 and the comparison between the calculated and observed shoreline change rates are shown in Figure A5. Table A4 shows shoreline change modeling statistics for the case.

1991-2000 setup

The model was also run for the 1991 to 2000 time period. Although validation cases should be run for separate time periods than the calibrated case, the 1991 to 2000 time period is the longest time period where wave data and initial shoreline are available. Since GenCade does not have the ability to create an inlet in the middle of a simulation, simulations that start before Mitchell's Cut was opened should not be used. Again, the 1991 shoreline does not extend the entire GenCade domain, so the 1990 shoreline was taken as the initial shoreline for approximately the northernmost 15,000 ft.

All of the model parameters, shoal volumes, sediment bypassing, and setup are identical to the 1995 to 2000 calibrated case. Data were taken from the same WIS hindcast stations from 1991 to the end of 1999.

Figure A6 shows a comparison between the calculated net longshore transport rates and those estimated and Figure A7 shows the comparison between the calculated and observed shoreline change rates.

Figure A4. Net transport, model results vs. published values for the 1991-1995 case.
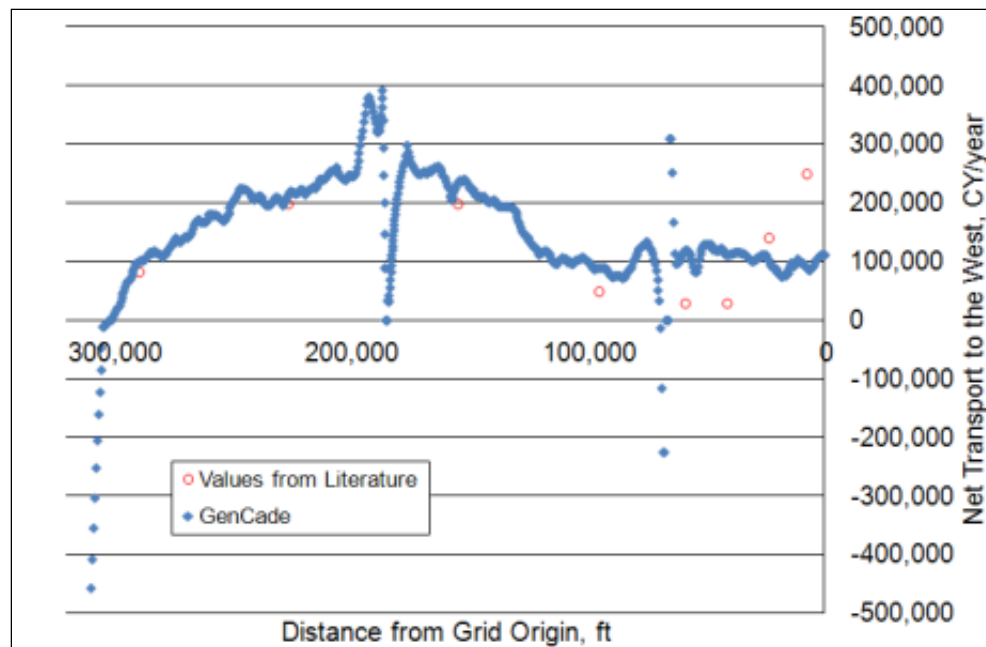

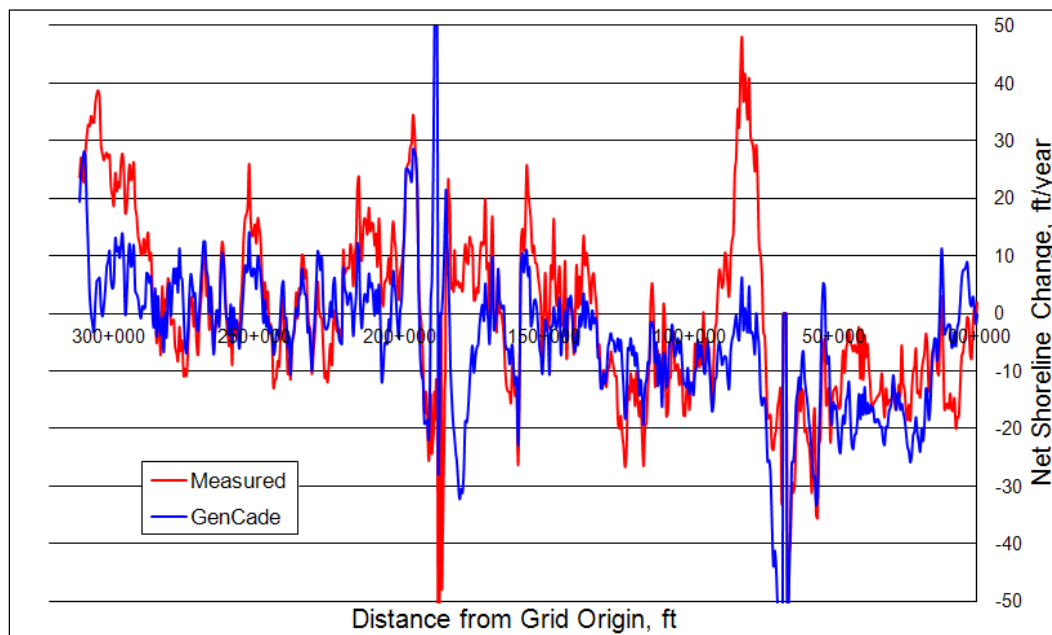Figure A5. Shoreline change, model results vs.published values for the 1991-1995 case.

Table A4. Shoreline change modeling statistics for the 1991-1995 case.

Cell	Average Shoreline Change, ft/yr		RMS Error, ft/yr	Brier Skill Score
	Measured	Modeled		
SBR to Cedar Lakes	-8.7	0.2	10.5	0.09
West of Cedar Lakes	-12.4	-18.1	7.2	0.69
Sargent: East of FM 457	-10.9	-17.9	8.7	0.47
Sargent: West of FM 457	-23.0	-19.1	6.1	0.93
West of Mitchell's Cut	-3.2	-11.4	16.4	0.16
East of MCR	3.6	-4.1	13.0	-0.60
MCR to MSC: North	2.9	4.4	16.7	-0.89
MCR to MSC: South	13.6	5.4	15.2	0.41

Figure A6. Net transport, model results vs. published values for the 1991-2000 case.
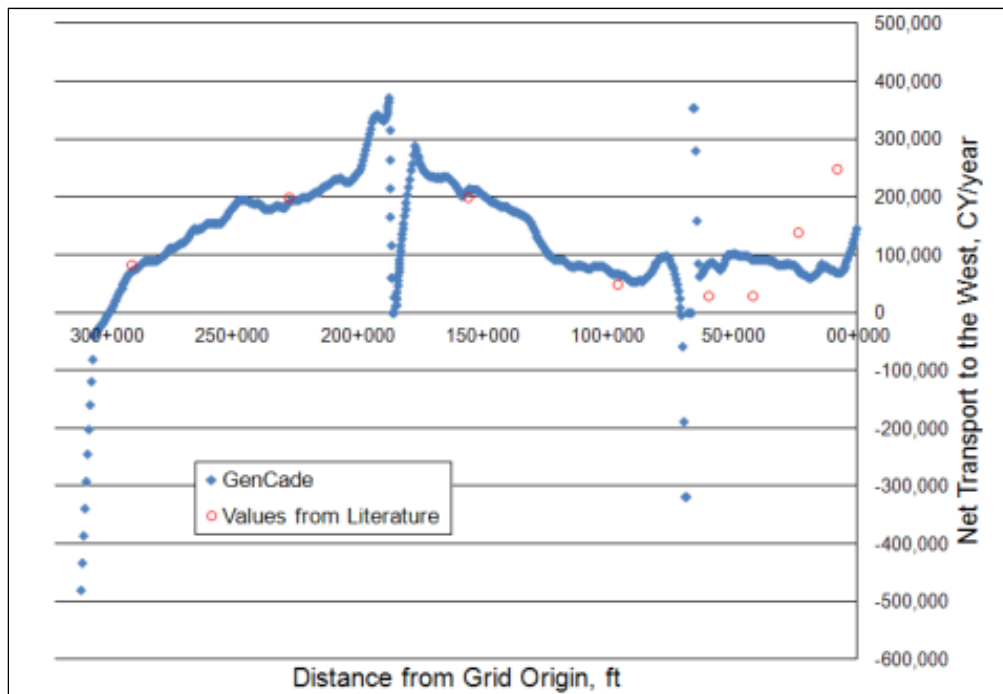
Figure A7. Shoreline change, model results vs. published values for the 1991-2000 case.
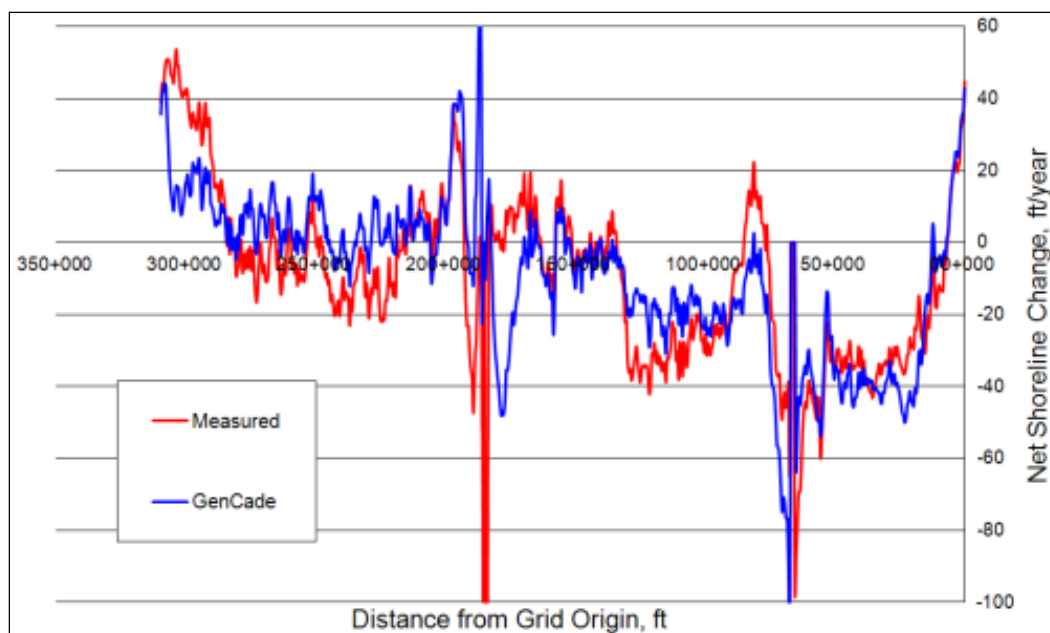

Table A5. Shoreline change modeling statistics for the 1991-2000 case.

Cell	Average Shoreline Change, ft/yr		RMS Error, ft/yr	Brier Skill Score
	Measured	Modeled		
SBR to Cedar Lakes	1.8	6.7	6.3	0.90
West of Cedar Lakes	-29.6	-39.7	11.3	0.86
Sargent: East of FM 457	-34.8	-39.3	6.0	0.97
Sargent: West of FM 457	-49.1	-36.8	12.1	0.94
West of Mitchell's Cut	-21.8	-23.9	13.7	0.74
East of MCR	-1.8	-9.5	17.2	-0.84
MCR to MSC: North	-4.3	8.1	19.1	-0.76
MCR to MSC: South	19.6	12.2	17.5	0.64

Groin alternatives near Mouth of Colorado River

Alternative 1: 400-ft groins, 800-ft spacing

Alternative 1 consists of seven groins at varying lengths and distances apart. The groins in the groin system are 400 ft long and are spaced 800 ft apart. This alternative consists of the shortest groins that are the closest together and also requires the greatest amount of groins. At the southern end of the groin system, there is a transition section that consists of three additional groins that are shorter and spaced closer together. The total distance of the entire groin system is 0.8 miles. The first groin for Alternatives 1-5 is located approximately one-half mile south of 3 Mile Cut. The locations and

lengths of the groins in this alternative are shown in Figure A8. In addition, all of the alternatives include a bypassing rate of 115,000 yd³/yr. Several of the alternatives were also simulated with minimal or no bypassing. After the longer 16-yr simulation, nearly 50 ft of sediment accumulates near the east jetty of MCR while nearly the same amount of sediment erodes from the west jetty. Figures A9 and A10 illustrate the results of net shoreline change for 5 and 16 yr.

Alternative 2: 400-ft groins, 1,200-ft spacing

Alternative 2 consists of five groins. The lengths of the groins in the groin field are also 400 ft like Alternative 1, but the spacing has increased to 1,200 ft. The final two groins are shorter and closer together than the constant length and spacing. The total distance from the first groin to the last groin is approximately 0.8 miles. The setup for Alternative 2 is shown in Figure A11. Figures A12 and A13 illustrate the results of net shoreline change for 5 and 16 yr.

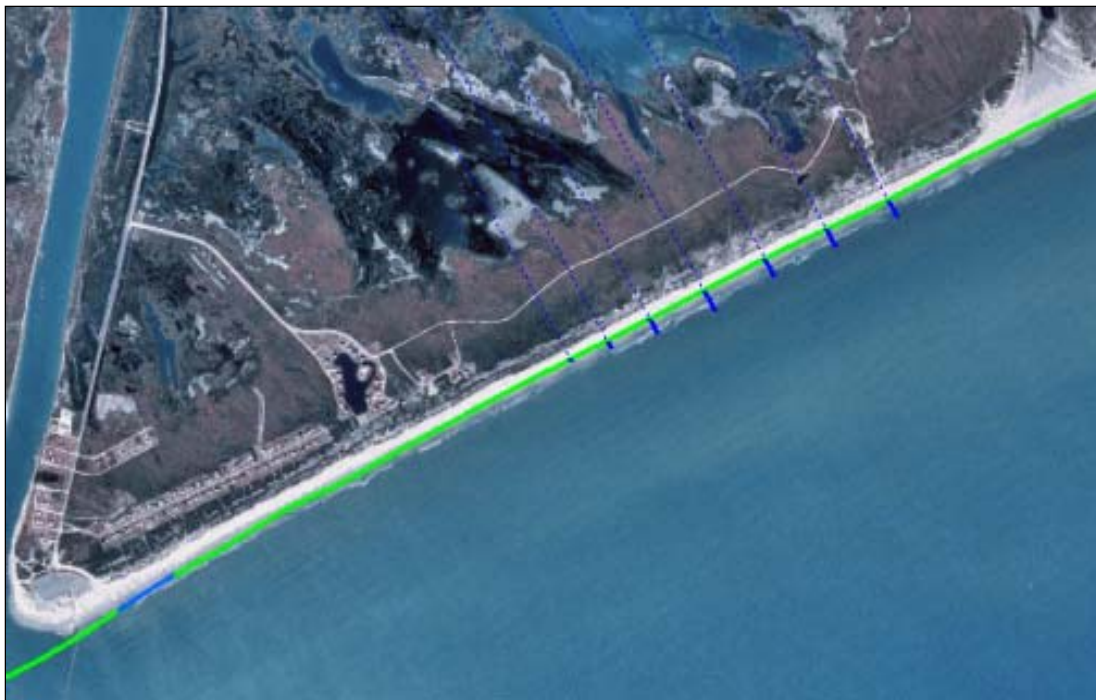Figure A8. GenCade setup between MCR and 3 Mile Cut for Alternative 1.

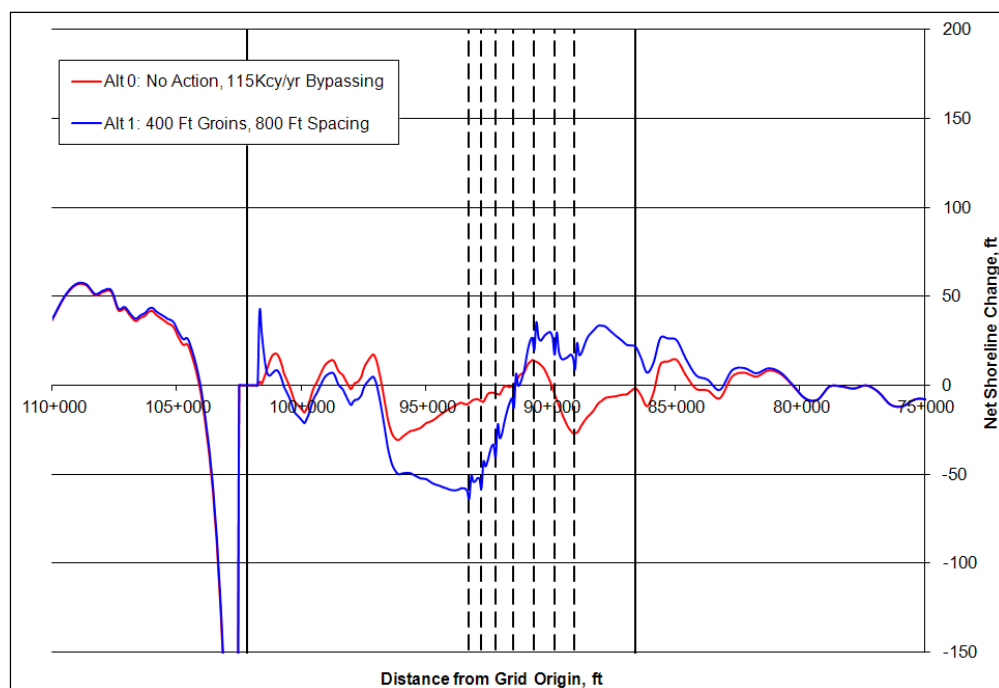Figure A9. Shoreline change for Alternatives 1 and 0 after 5 yr.

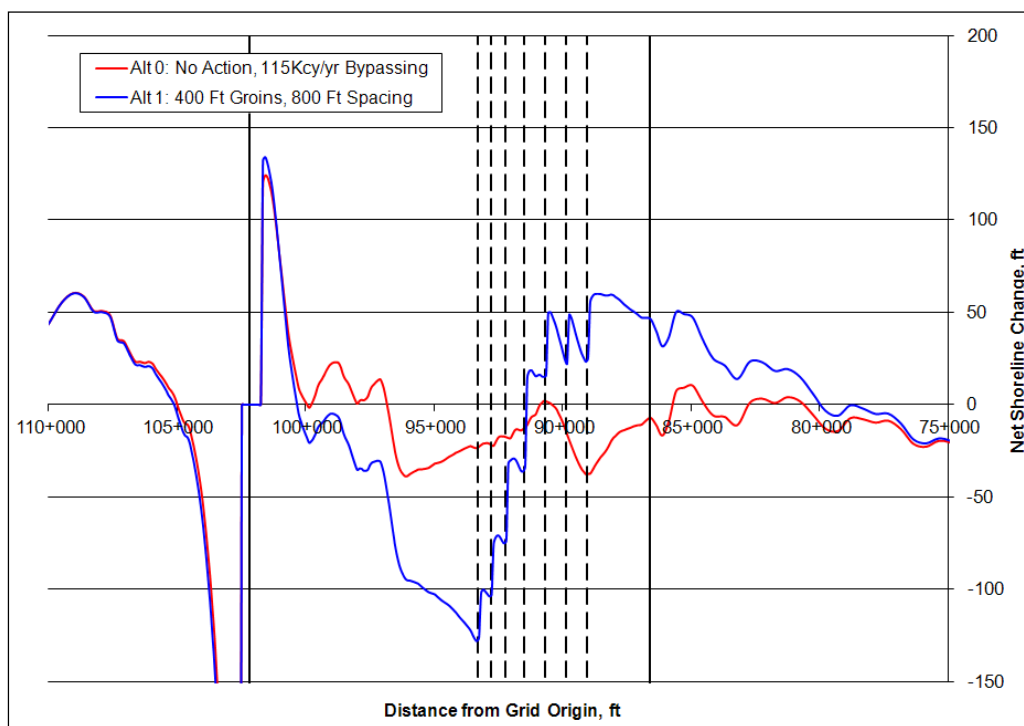Figure A10. Shoreline change for Alternatives 1 and 0 after 16 yr.

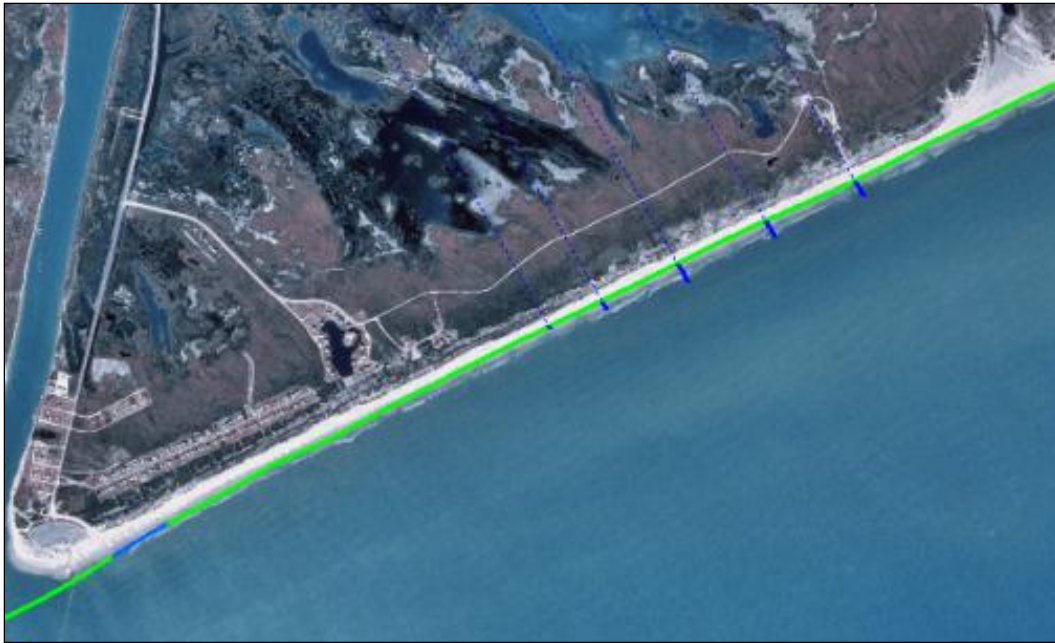Figure A11. GenCade setup between MCR and 3 Mile Cut for Alternative 2.

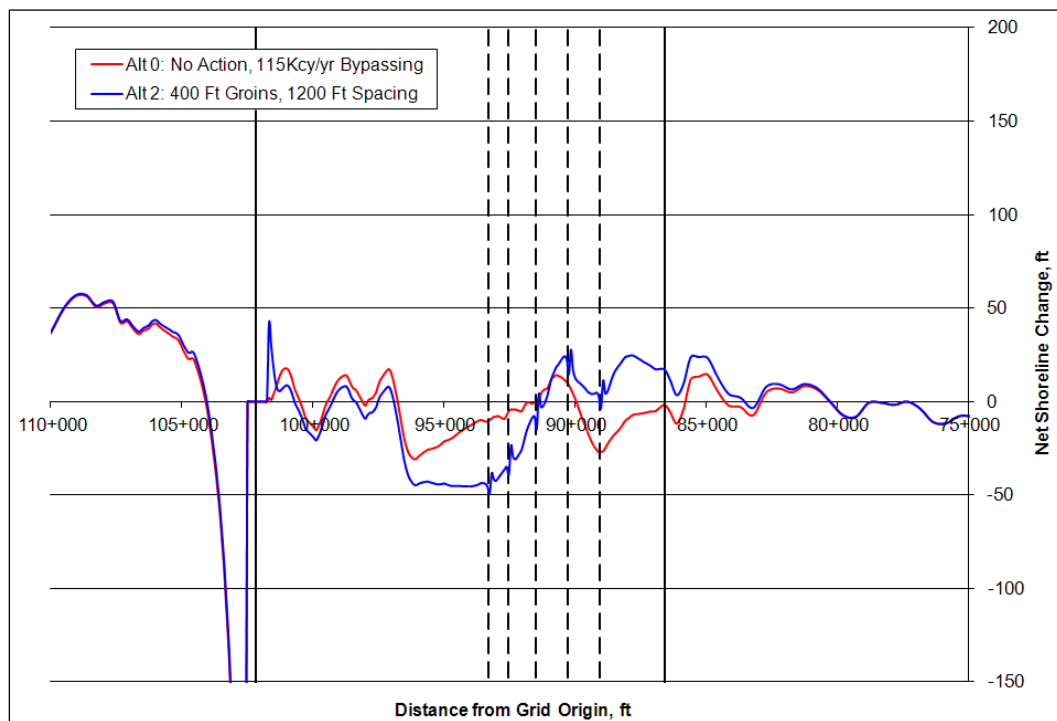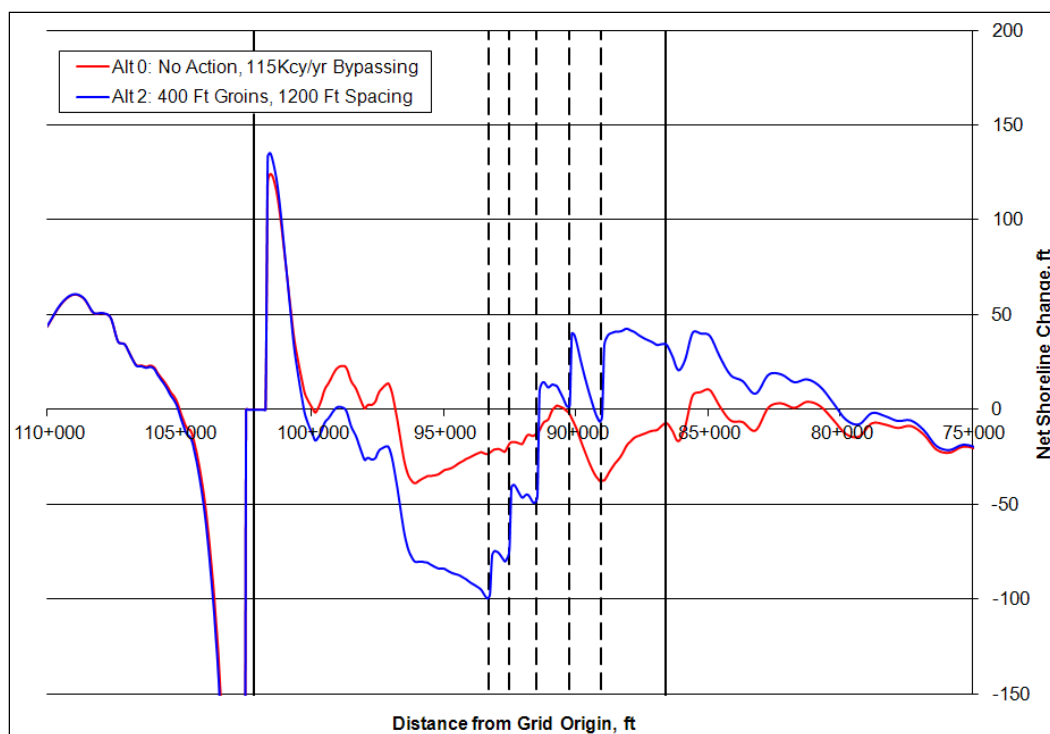Figure A12. Shoreline change for Alternatives 2 and 0 after 5 yr.

Figure A13. Shoreline change for Alternatives 2 and 0 after 16 yr.

Alternative 3: 600-ft groins, 1,200-ft spacing

There are five groins in Alternative 3. The first three groins are 600 ft long and 1,200 ft apart. The final two groins are shorter and closer together than the first three groins. The total distance of the groin system is about 0.83 miles. This setup is shown in Figure A14. Figures A15 and A16 show the results of net shoreline change for 5 and 16 yr.

Alternative 4: 600-ft groins, 1,800-ft spacing

Alternative 4 consists of five groins. The first three groins are 600 ft and spaced 1,800 ft apart. The final two groins are shorter and closer together. The total distance of the groin system is 1.2 miles long. This setup is shown in Figure A17. Figures A18 and A19 show the results of net shoreline change for 5 and 16 yr.

Alternative 5: 800-ft groins, 1,600-ft spacing

Since none of the previously discussed alternatives resulted in the 200-ft shoreline advance requested, Alternative 5 was designed. Three groins of 800 ft were spaced 1,600 ft apart for this alternative. Since only three groins were used, there is no transition spacing. The total length of the groin field

is about 0.6 mi. It is expected that the longer groins will cause more accretion to the east and more erosion to the west of the groin field. Figure A20 shows the setup of Alternative 5. Figures A21 and A22 illustrate the results of net shoreline change for 5 and 16 yr.

Figure A14. GenCade setup between MCR and 3 Mile Cut for Alternative 3.

Figure A15. Shoreline change for Alternatives 3 and 0 after 5 yr.

Figure A16. Shoreline change for Alternatives 3 and 0 after 16 yr.

Figure A17. GenCade setup between MCR and 3 Mile Cut for Alternative 4.
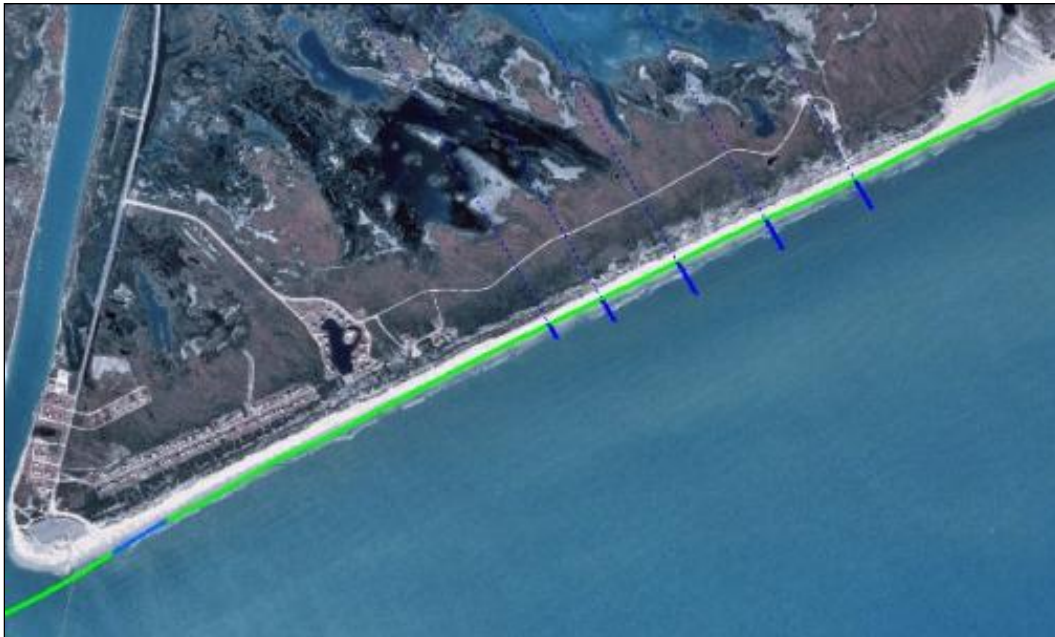
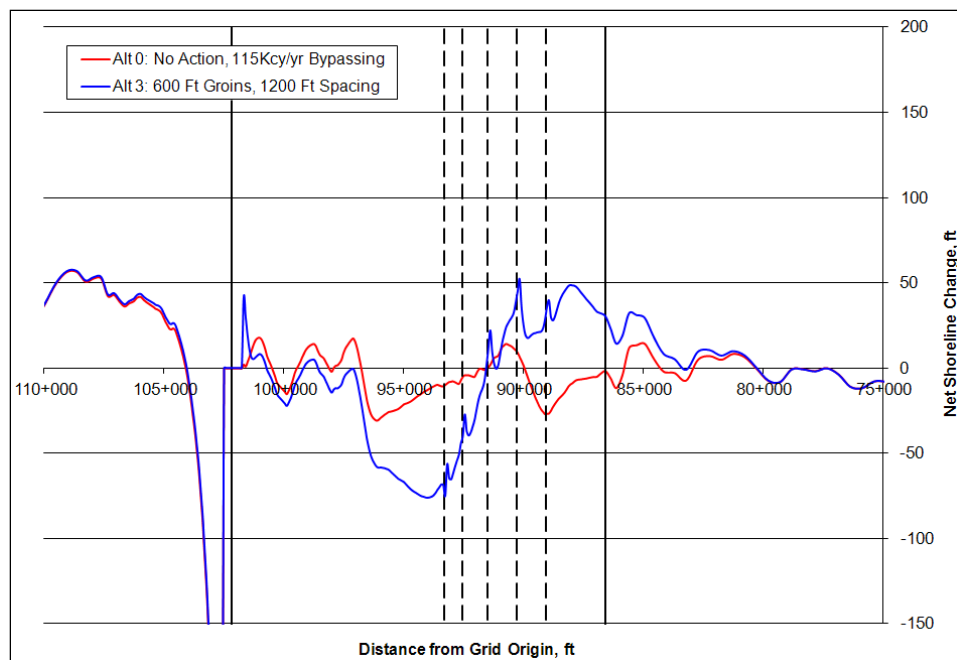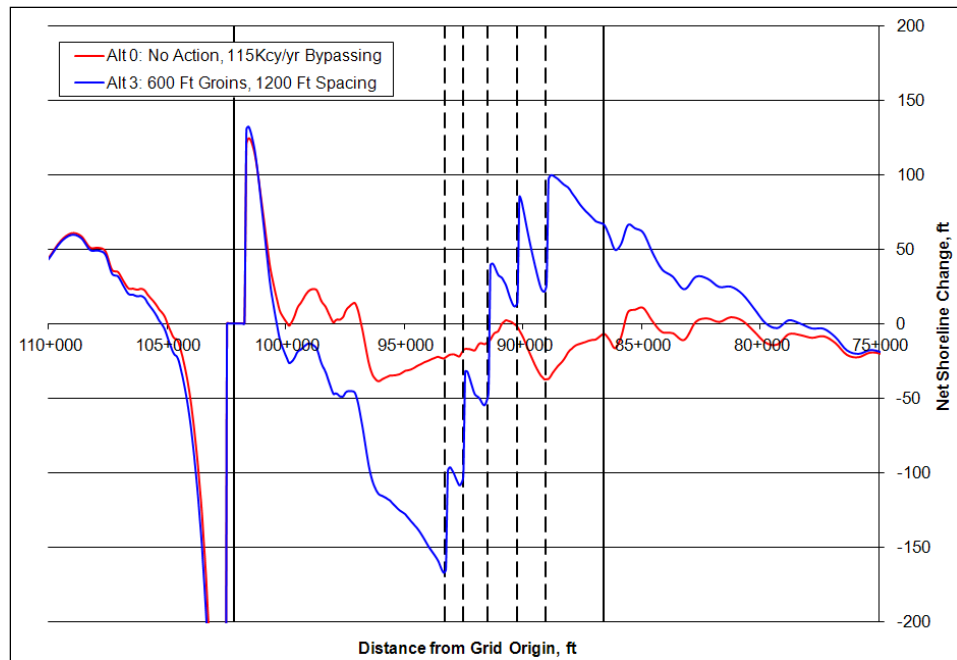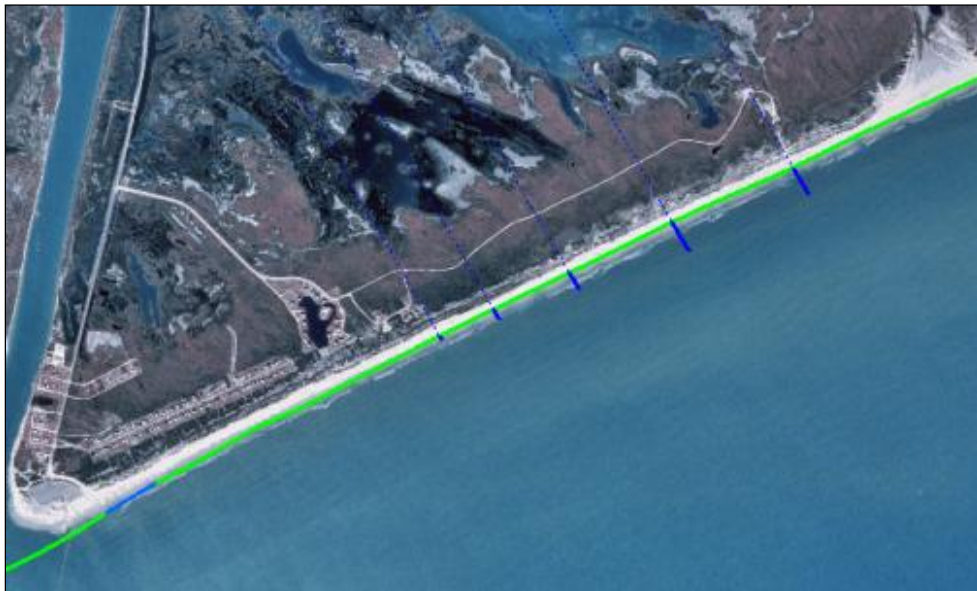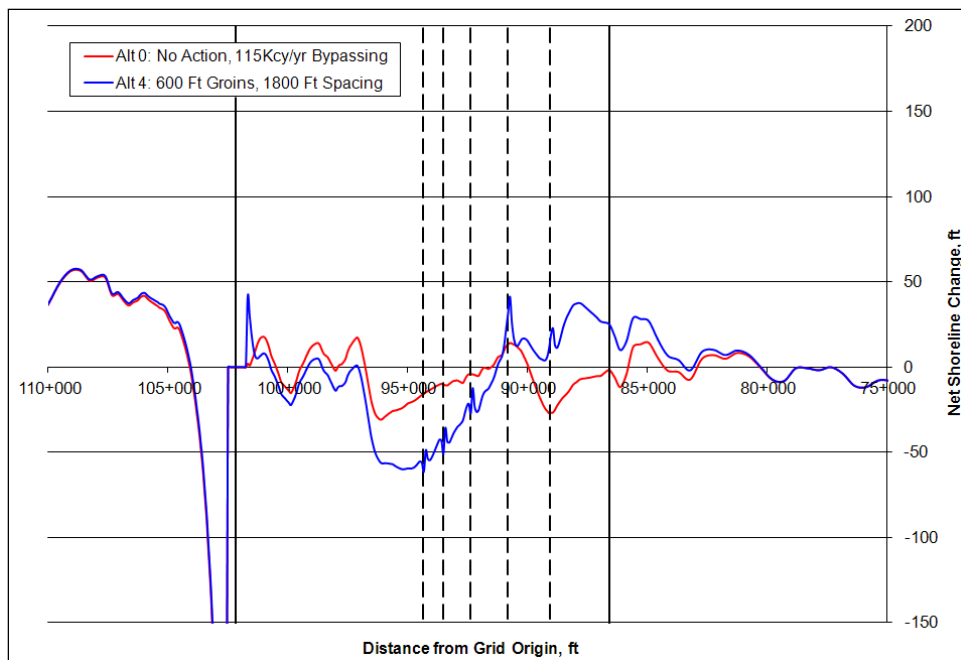Figure A18. Shoreline change for Alternatives 4 and 0 after 5 yr.

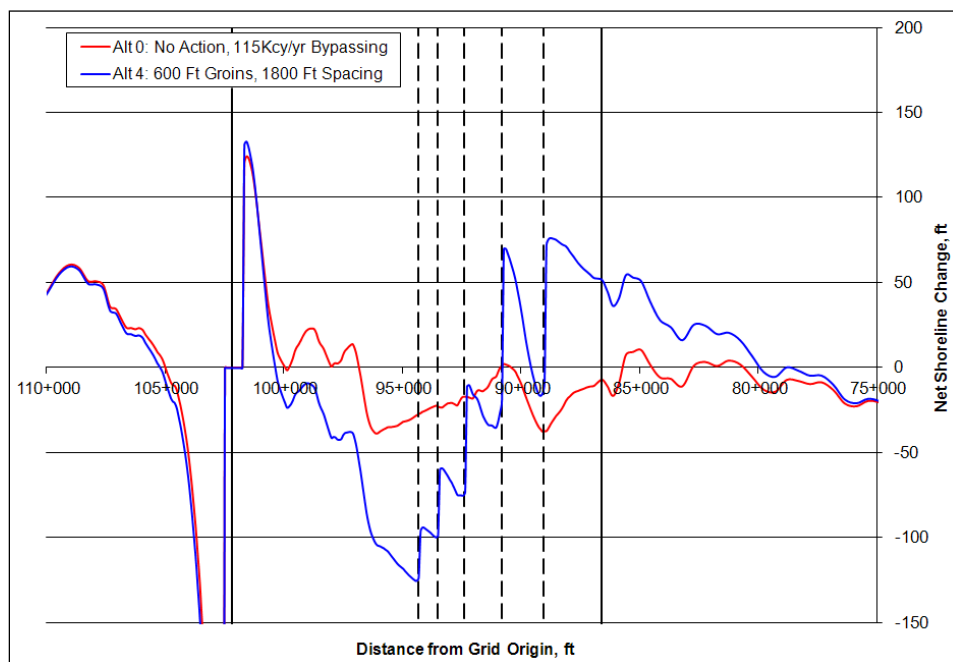Figure A19. Shoreline change for Alternatives 4 and 0 after 16 yr.

Figure A20. GenCade setup between MCR and 3 Mile Cut for Alternative 5.

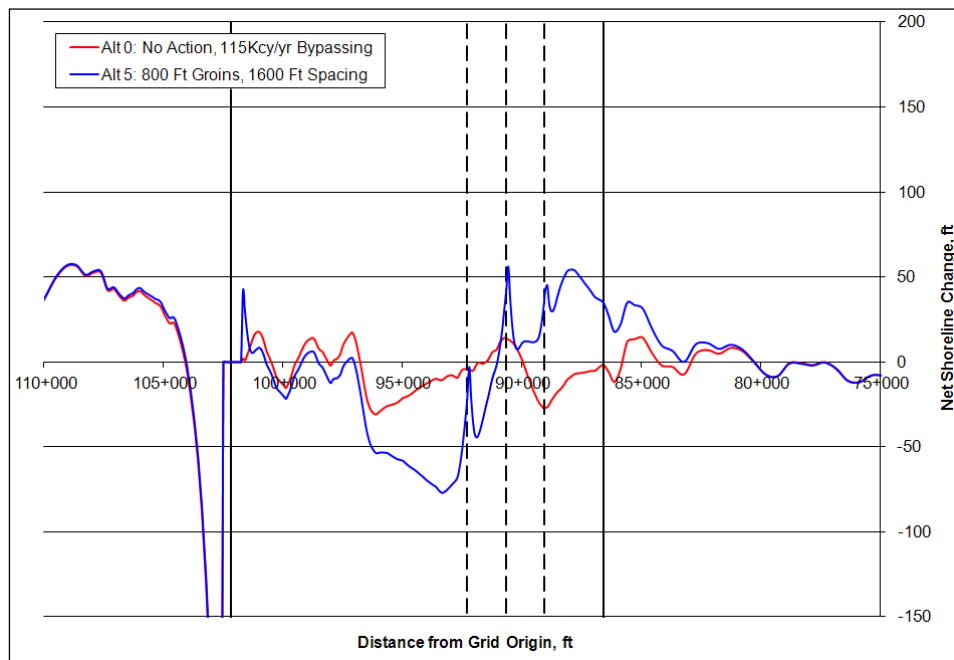Figure A21. Shoreline change for Alternatives 5 and 0 after 5 yr.

Figure A22. Shoreline change for Alternatives 5 and 0 after 16 yr.
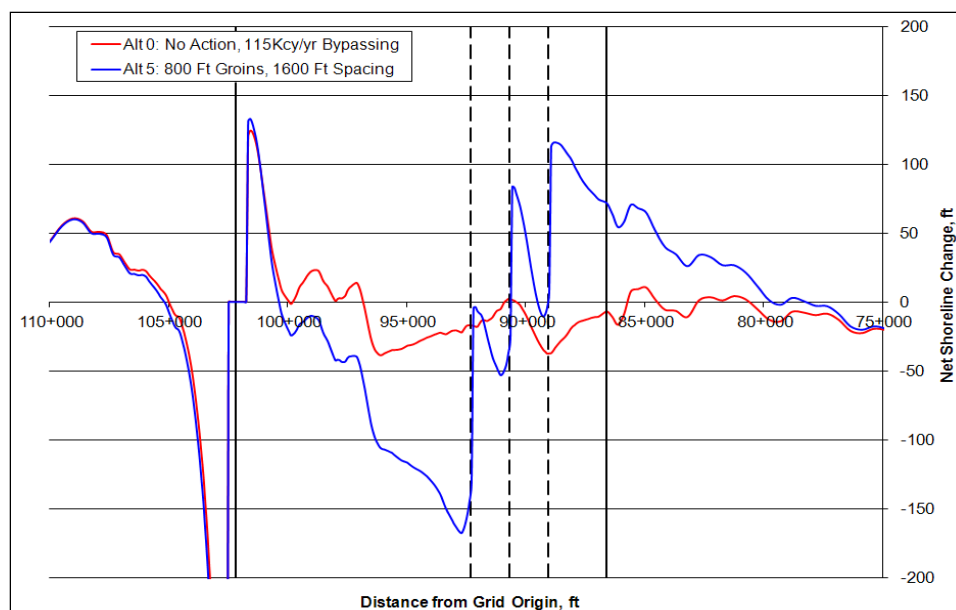
Alternative 5: 800-ft groins, 1,600-ft spacing with beach fill

None of the first four alternatives resulted in the desired 200 ft of beach advance. Therefore, Alternative 5 was modified to include a beach fill. The groins in Alternative 5 with beach fill are identical to the groin setup in Alternative 5. This beach fill consists of 243,000 yd³ of sediment placed over the distance between the groins. This should provide about 100 ft of added berm width. More material could be added or the material could be spread out over a longer distance. Figures A23 and A24 show the results of net shoreline change for 5 and 16 yr.

Alternative 1 - Shifted: 400-ft groins, 800-ft spacing

All of the alternatives resulted in some amount of sand accretion near 3 Mile Cut. Therefore, all of the alternatives were rerun with the groin field shifted approximately 1640 ft (500 m) to the southwest. The first groin in all of the shifted alternatives is about 0.75 miles west of 3 Mile Cut. The configurations of all of the alternatives are identical to the previous alternatives. For Alternative 1 - Shifted, the groins are the same length and spaced the same distance apart as Alternative 1, but the first groin is 1,640 ft (500 m) southwest of the first groin in Alternative 1. As is the case in Alternative 1, Alternative 1 – Shifted consists of seven 400-ft groins that are spaced 800 ft apart. The last three groins are spaced closer together and are shorter. The groin configuration of Alternative 1 – Shifted is shown in Figure A25. Figures A26 and A27 illustrate the results of net shoreline change for 5 and 16 yr.

Figure A23. Shoreline change for Alternative 5 – shifted with beach fill and Alternative 0 after 5 yr.
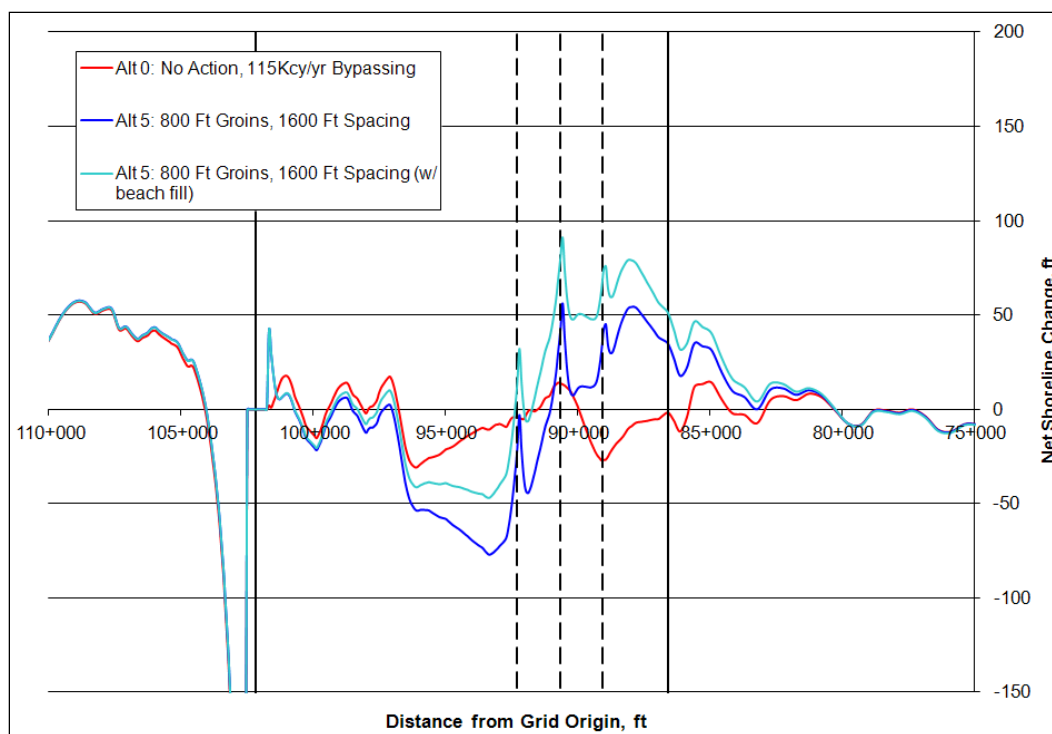
Figure A24. Shoreline change for Alternative 5 - shifted with beach fill and Alternative 0 after 16 yr.

Figure A25. GenCade setup between MCR and 3 Mile Cut for Alternative 1 - shifted.

Figure A26. Shoreline change for Alternative 1 - shifted and Alternative 0 after 5 yr.

Figure A27. Shoreline change for Alternative 1 - shifted and Alternative 0 after 16 yr.
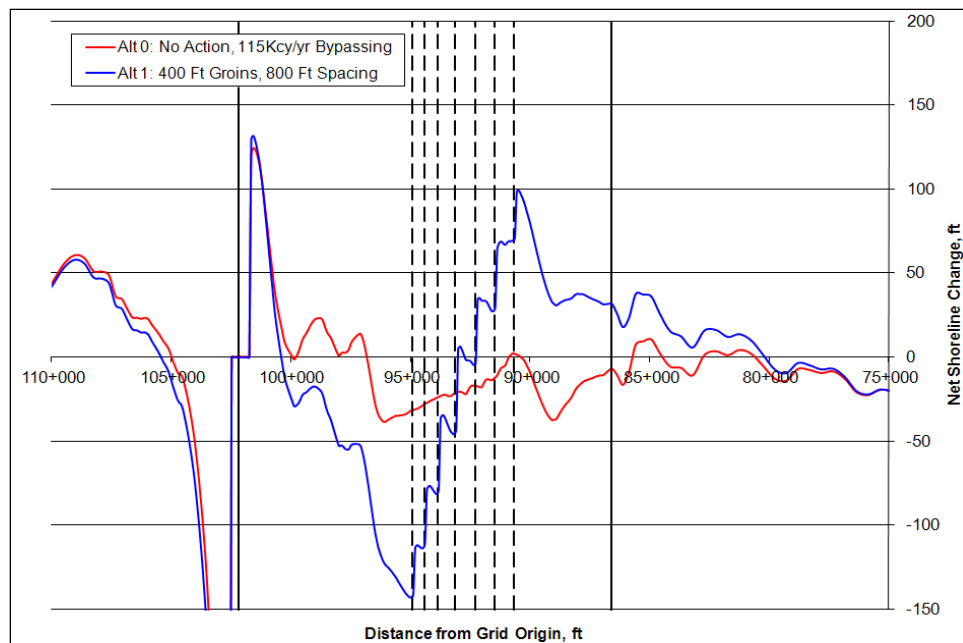
Alternative 2 - Shifted: 400-ft groins, 1,200-ft spacing

The groin configuration for Alternative 2 – Shifted is identical to Alternative 2. There are five groins of 400 ft that are spaced 1,200 ft apart. The last two groins are closer together and shorter. The first groin is 1,640 ft further southwest than the first groin in Alternative 2. Figure A28 shows the setup, and Figures A29 and A30 illustrate the results of net shoreline change for 5 and 16 yr.

Figure A28. GenCade setup between MCR and 3 Mile Cut for Alternative 2 - shifted.

Figure A29. Shoreline change for Alternative 2 - Shifted and Alternative 0 after 5 yr.

Figure A30. Shoreline change for Alternative 2 - Shifted and Alternative 0 after 16 yr.

Alternative 3 - Shifted: 600-ft groins, 1,200-ft spacing

Alternative 3 – Shifted consists of five 600-ft groins that are spaced 1,200 ft apart. The last two groins are shorter and are spaced closer together. The setup (Figure A31) is the same as Alternative 3, but the first groin is located about 1,640 ft further southwest. Figures A32 and A33 show the results of net shoreline change for 5 and 16 yr.

Figure A31. GenCade setup between MCR and 3 Mile Cut for Alternative 3 - shifted.

Figure A32. Shoreline change for Alternative 3 - shifted and Alternative 0 after 5 yr.

Figure A33. Shoreline change for Alternative 3 - shifted and Alternative 0 after 16 yr.

Alternative 4 - Shifted: 600-ft groins, 1,800-ft spacing

Alternative 4 – Shifted consists of the same configuration as Alternative 4 (600- ft groins spaced 1,800 ft apart). The last two of the five groins are shorter and closer together. The first groin is located about 0.75 miles south of the 3 Mile Cut region as shown in Figure A34. Figures A35 and A36 illustrate the results of net shoreline change for 5 and 16 yr.
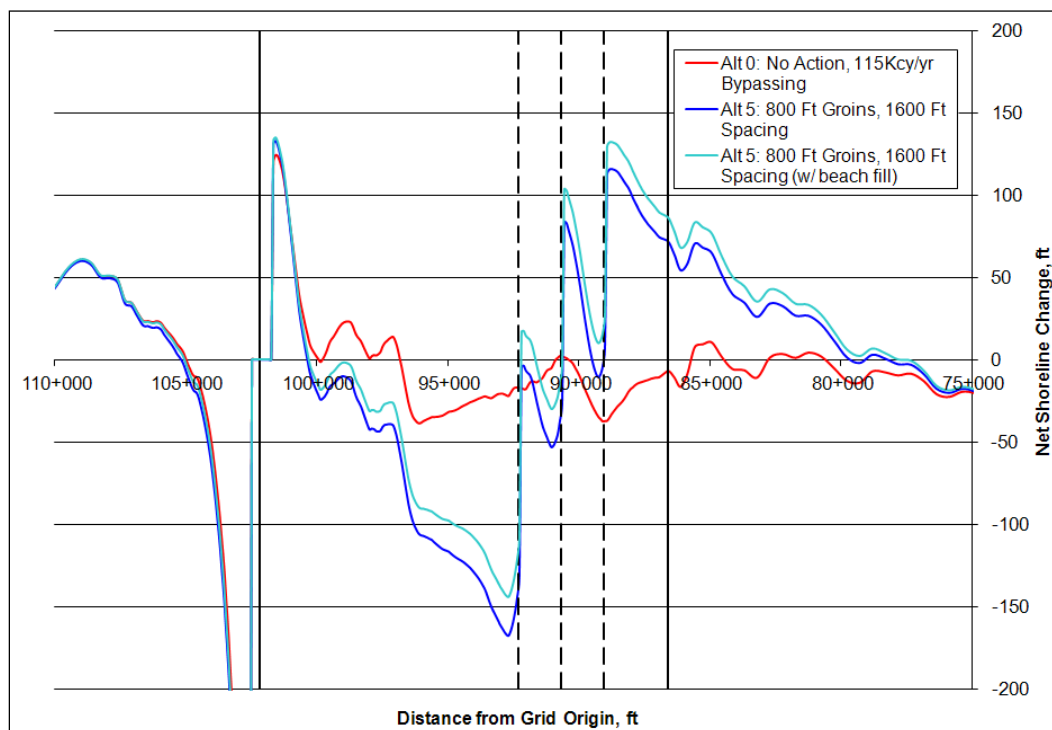
Figure A34. GenCade setup between MCR and 3 Mile Cut for Alternative 4 - shifted.

Figure A35. Shoreline change for Alternative 4 - shifted and Alternative 0 after 5 yr.
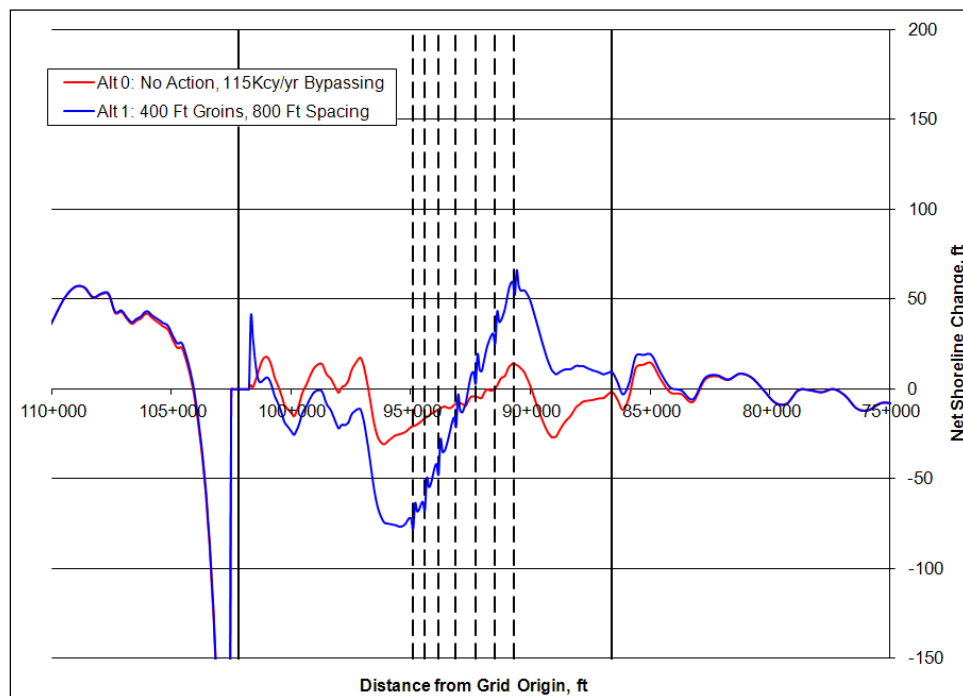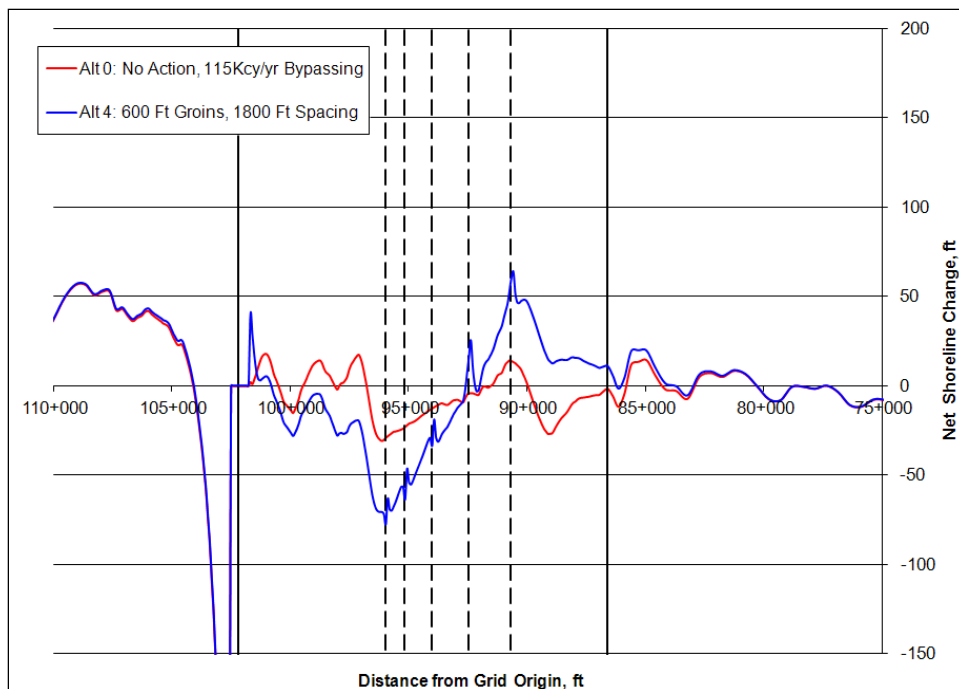

Figure A36. Shoreline change for Alternative 4 - shifted and Alternative 0 after 16 yr.
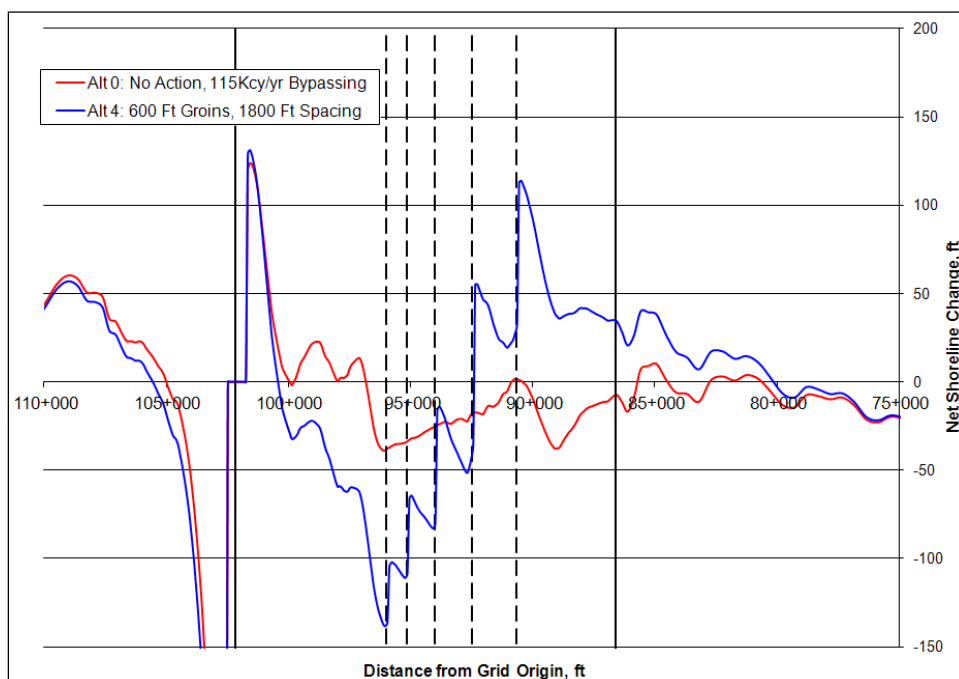
Alternative 5 - Shifted: 800-ft groins, 1,600-ft spacing

Alternative 5 – Shifted consists of three 800-ft groins spaced 1,600 ft apart. Since only three groins are used, the spacing between the groins is constant at 1,600 ft. This is the same configuration as Alternative 5, except the groin

field has been shifted about 1640 ft to the southwest as shown in Figure A37. Figures A38 and A39 show the results of net shoreline change for 5 and 16 yr.

Figure A37. GenCade setup between MCR and 3 Mile Cut for Alternative 5 - shifted.

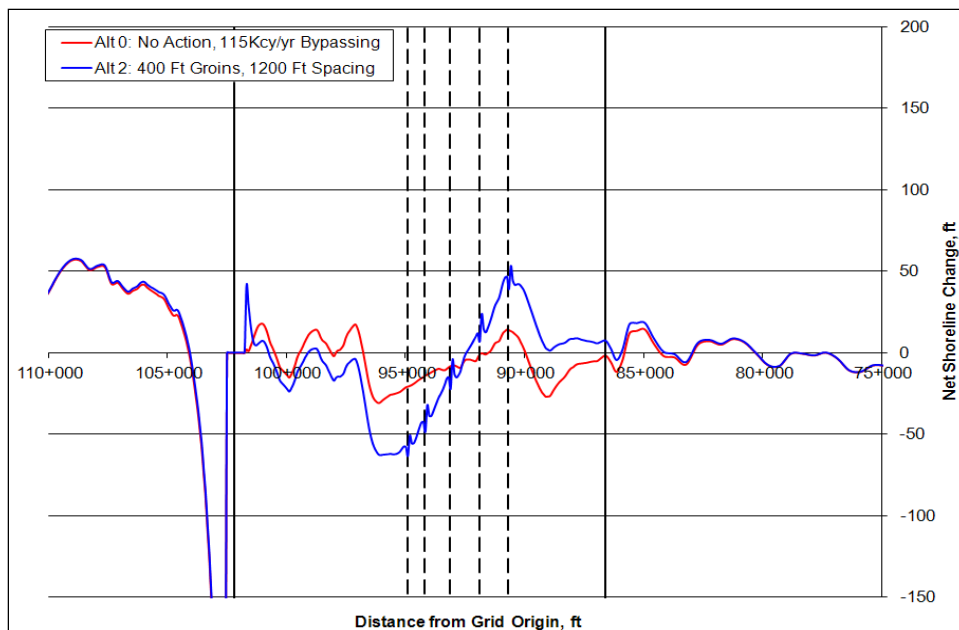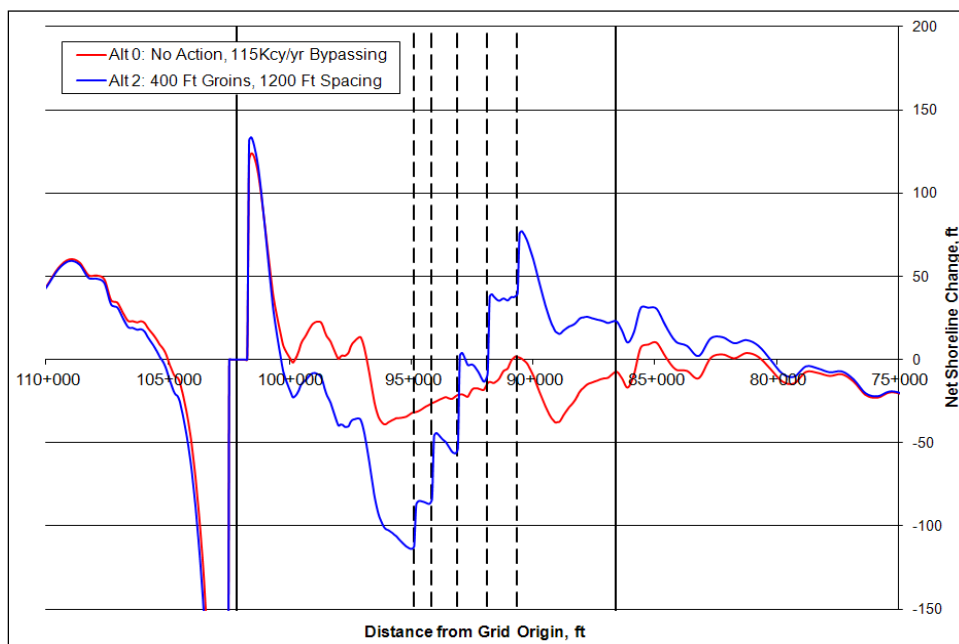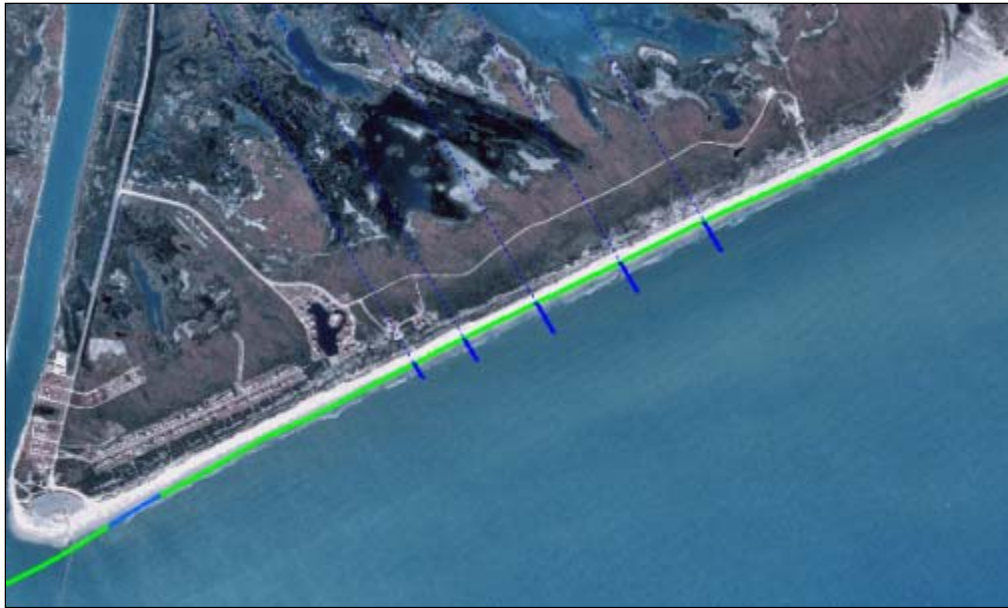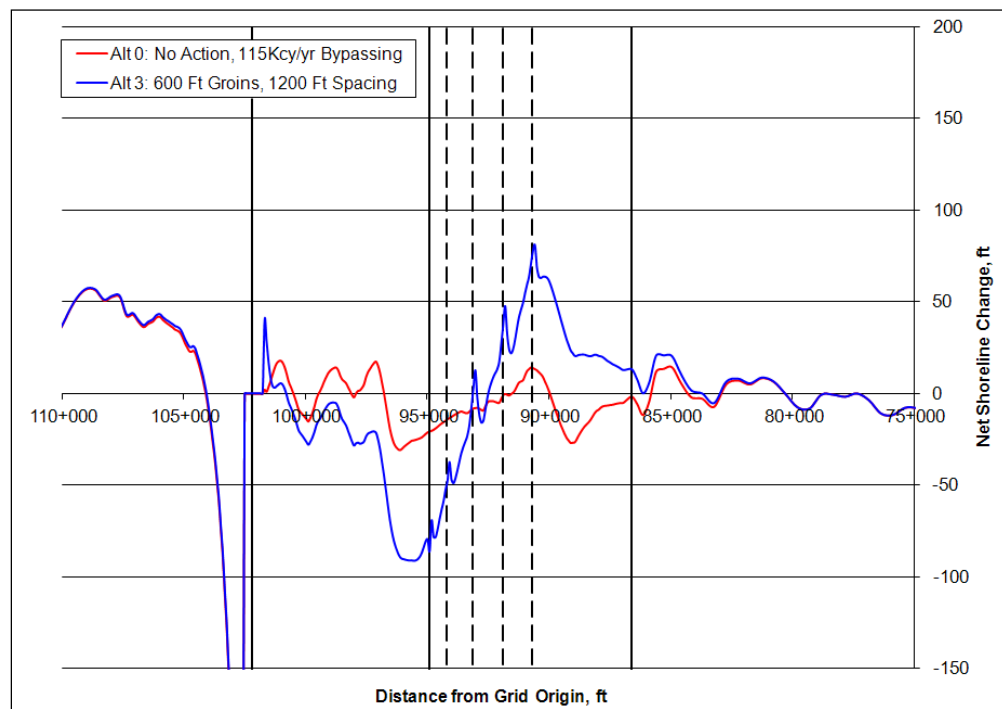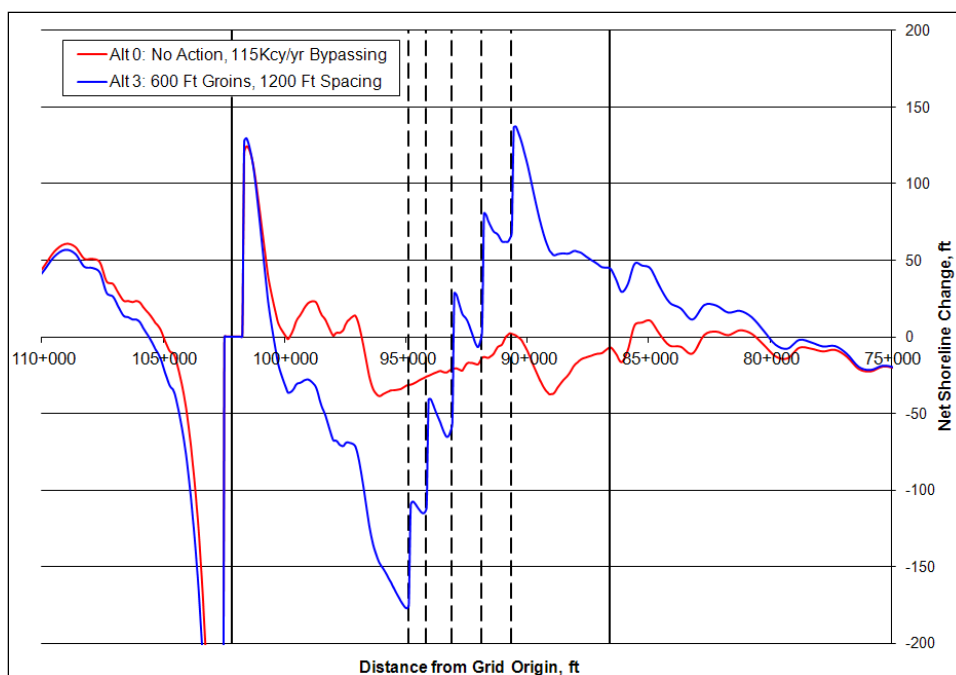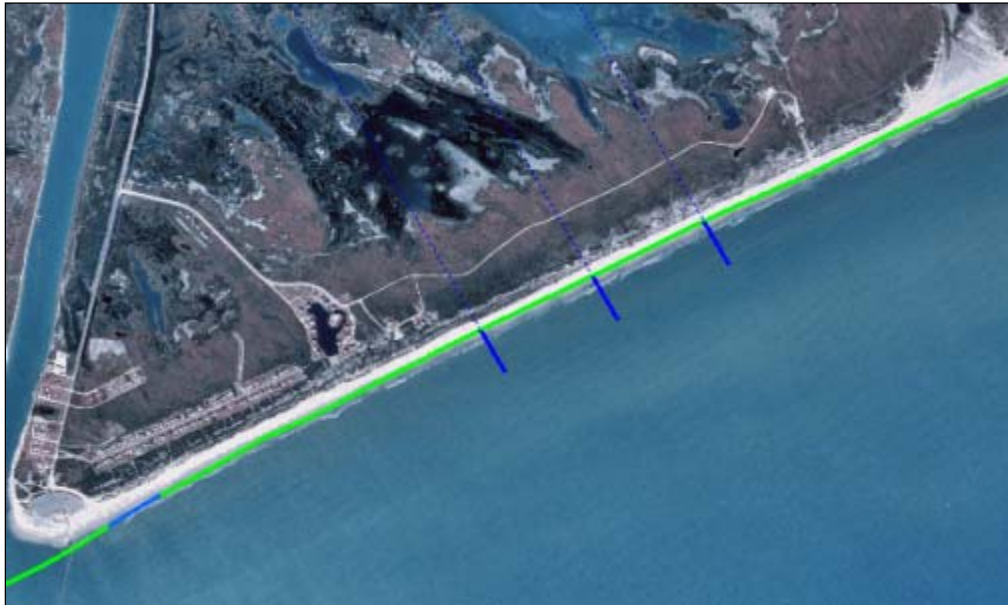Figure A38. Shoreline change for Alternative 5 - shifted and Alternative 0 after 5 yr
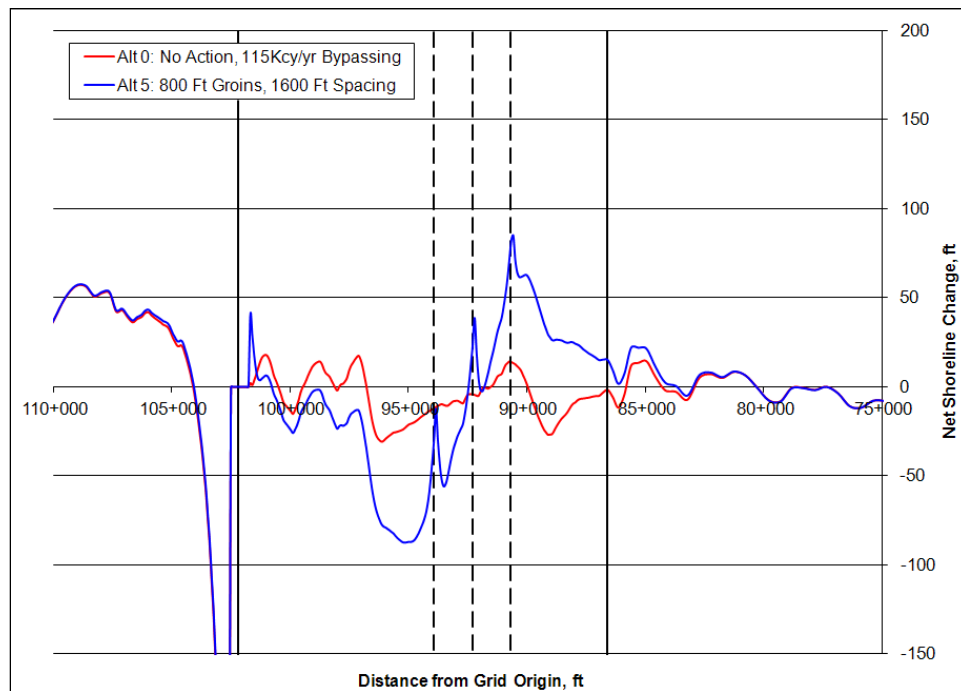
Figure A39. Shoreline change for Alternative 5 - shifted and Alternative 0 after 16 yr.
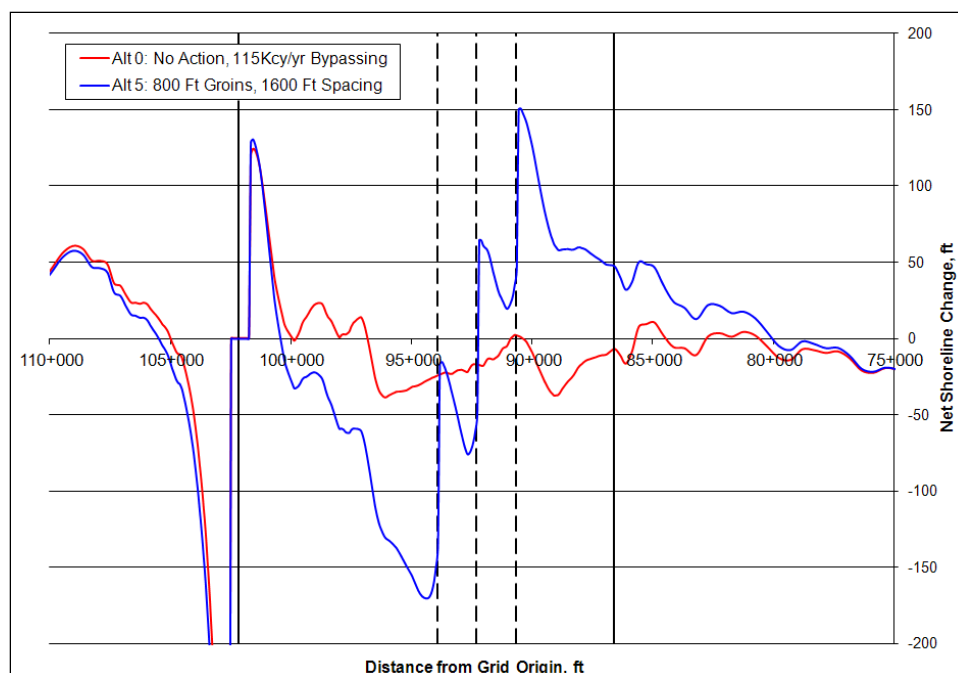
Alternative 5 – Shifted with beach fill: 800-ft groins, 1,600-ft spacing

None of the alternatives provided the recommended 200-ft shoreline advance, so Alternative 5 – Shifted was modified to add a beach fill. This beach fill consists of 243,000 yd3 of sediment placed over the distance between the groins. This should provide about 100 ft of added berm width. More material could be added or the material could be spread out over a longer distance. Figures A40 and A41 illustrate the results of net shoreline change for 5 and 16 yr.

Alternative 5 shifted – porosity sensitivity

Although a porosity of 0.3 was chosen for the final design, porosities of 0.45, 0.6, and 0.8 were also tested. The next four figures compare the results after 5 yr. Figure A42 compares different porosities when no bypassing occurs and no beach fill is constructed. As the porosity increases, the effects of the groins become less. The shoreline change when the groins have porosities of 0.8 is almost identical to Alternative 0. Figure A43 compares all porosities when a 200 ft wide beach fill is constructed but no mechanical bypassing occurs. All of the cases in Figure A43 predict more shoreline advance than Figure A42 since they are identical except for the added beach fill. Figure A44 illustrates each porosity when the bypassing rate is 200,000 yd3/yr and no beach fill is constructed. These scenarios

Figure A40. Shoreline change for Alternative 5 – shifted and Alternative 5 – shifted with beach fill and Alternative 0 after 5 yr.
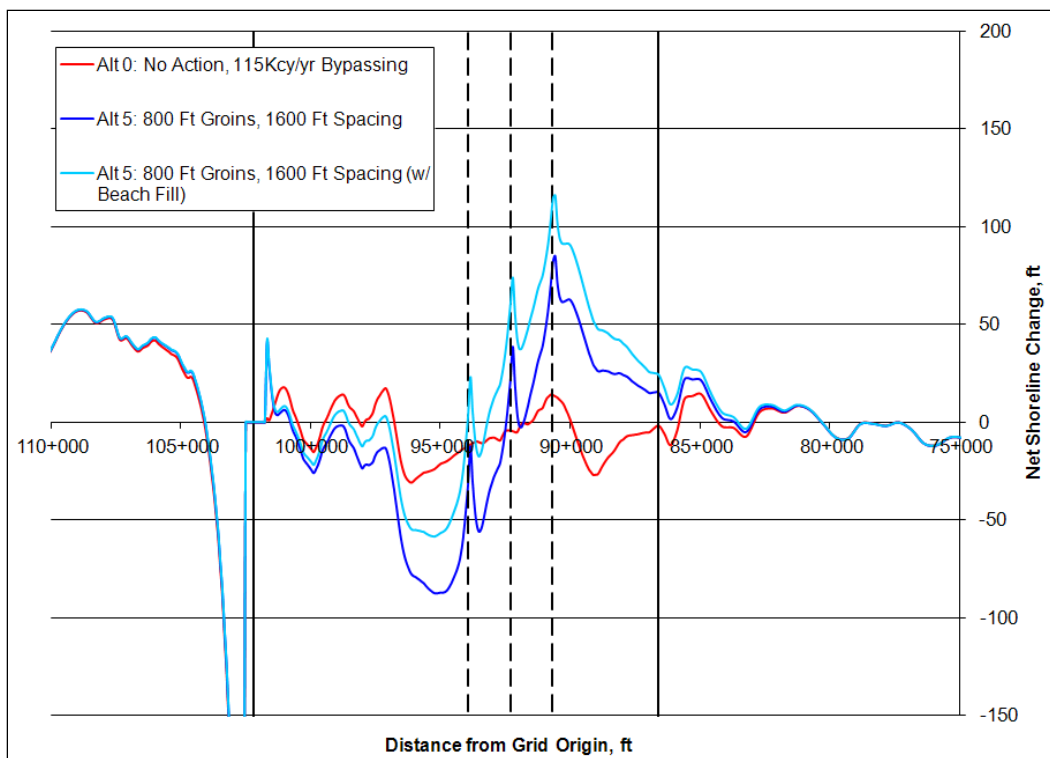
Figure A41. Shoreline change for Alternative 5 – shifted and Alternative 5 – shifted with beach fill and Alternative 0 after 16 yr.
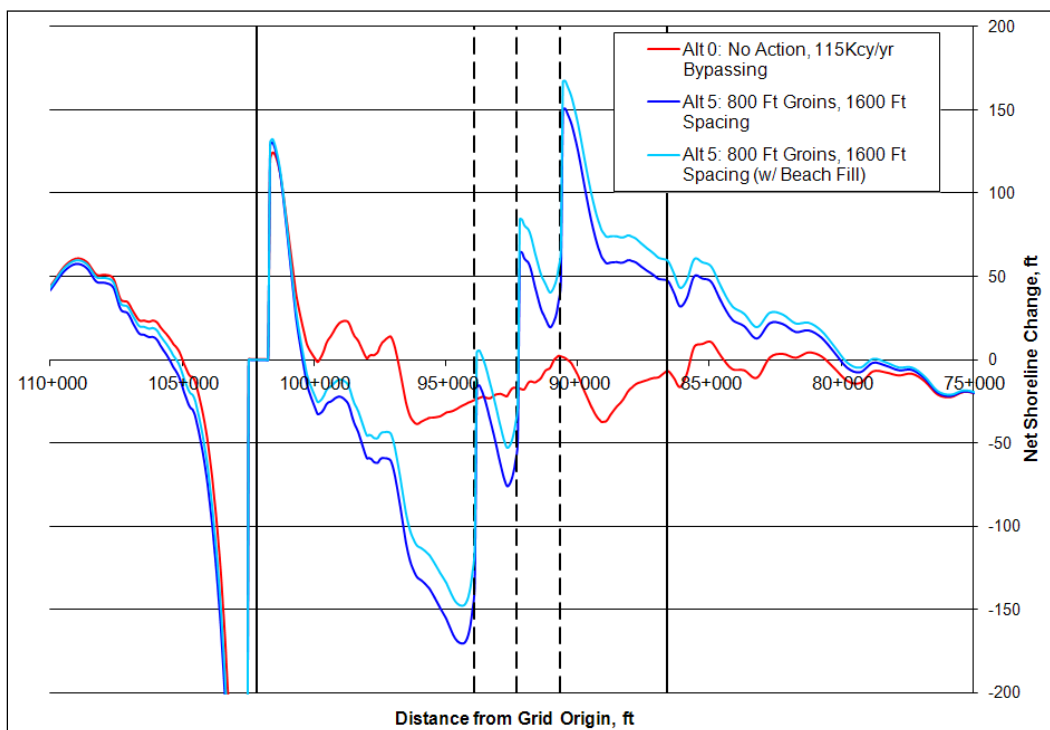
Figure A42. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 5 yr with no bypassing and no beach fill constructed.
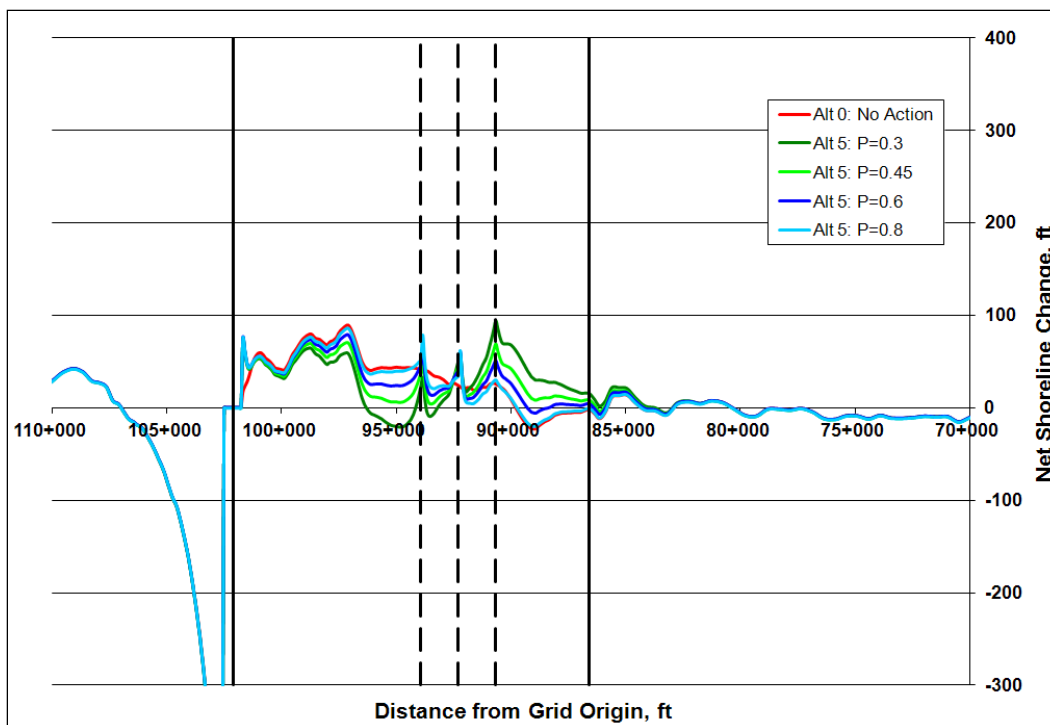
Figure A43. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 5 yr with no bypassing and a 200-ft wide beach fill.
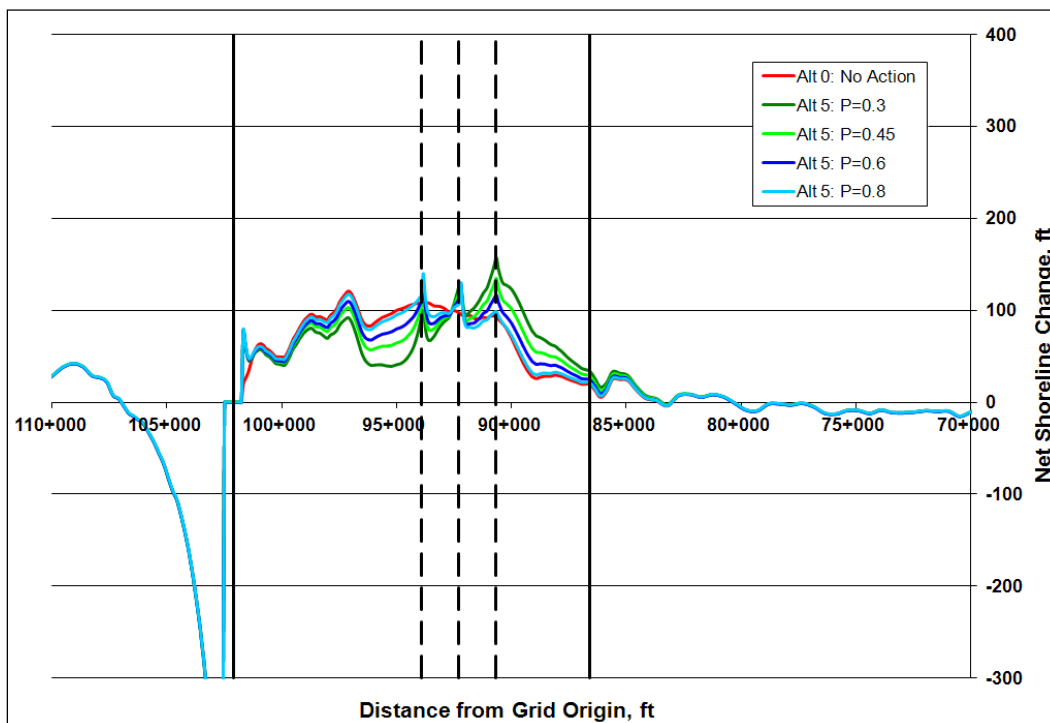
Figure A44. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 5 yr with 200,000 yd³/yr and no beach fill constructed.
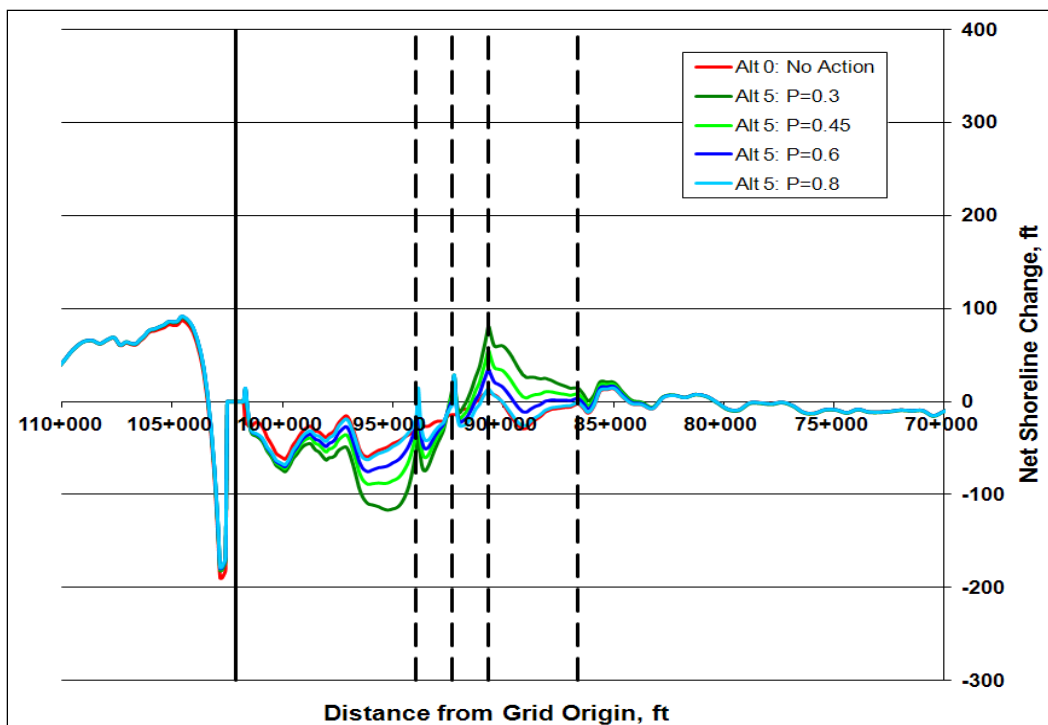
result in the greatest erosion southwest of the final groin. Shoreline change for each porosity when a 200-ft wide beach fill is constructed and the bypassing rate is 200,000 yd³/yr is shown in Figure A45. While the shoreline advance for all porosities is identical to Figure A43, the cases in Figure A45 result in more erosion southwest of the last groin. This is directly related to the amount of mechanical bypassing. If bypassing does not occur, then sediment will build up at the northeast jetty.

The next four figures show the same scenarios after 16 yr. Figure A46 shows shoreline change after 16 yr when no bypassing occurs and no beach fill is constructed while Figure A47 shows shoreline change with a bypassing rate of 0 yd³/yr and when a 200-ft wide beach fill is constructed. Figure A48 illustrates shoreline change after 16 yr with a bypassing rate of 200,000 yd³/yr but no beach fill is constructed while the shoreline change after 16 yr when a bypassing rate of 200,000 yd³/yr occurs and a 200 ft wide beach fill is constructed is shown in Figure A49. The figures are provided to show the overall picture of shoreline change between 3 Mile Cut and MCR. No additional discussion is included here, although these cases are included in the summary table.

Figure A45. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 5 yr with 200,000 yd³/yr and a 200-ft wide beach fill.
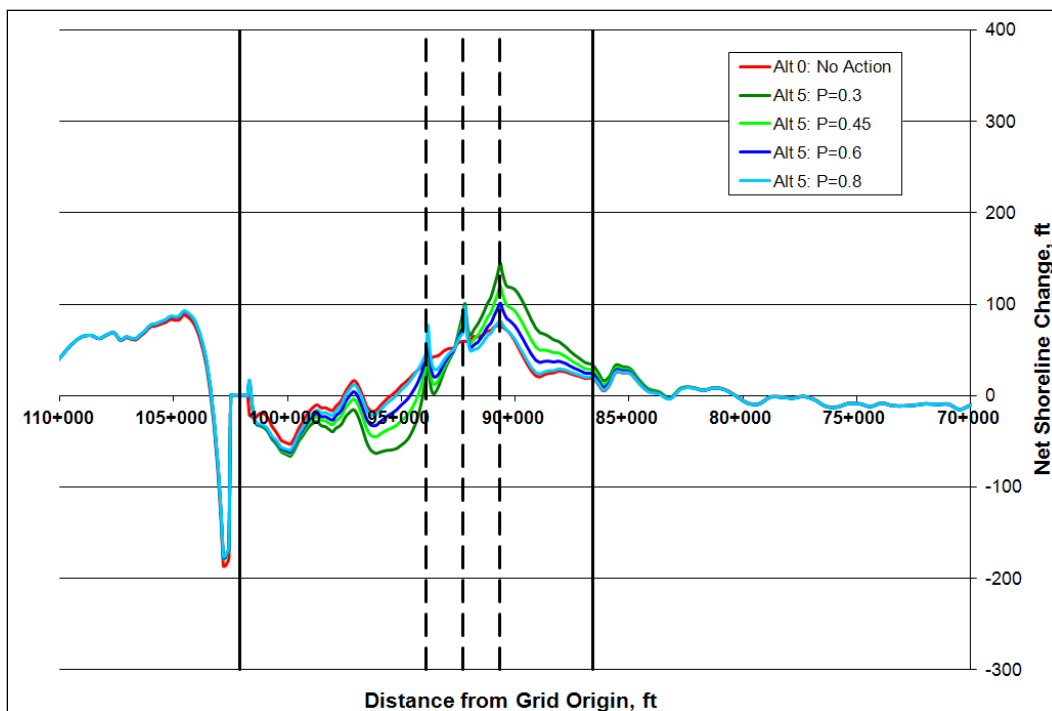
Figure A46. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 16 yr with no bypassing and no beach fill constructed.
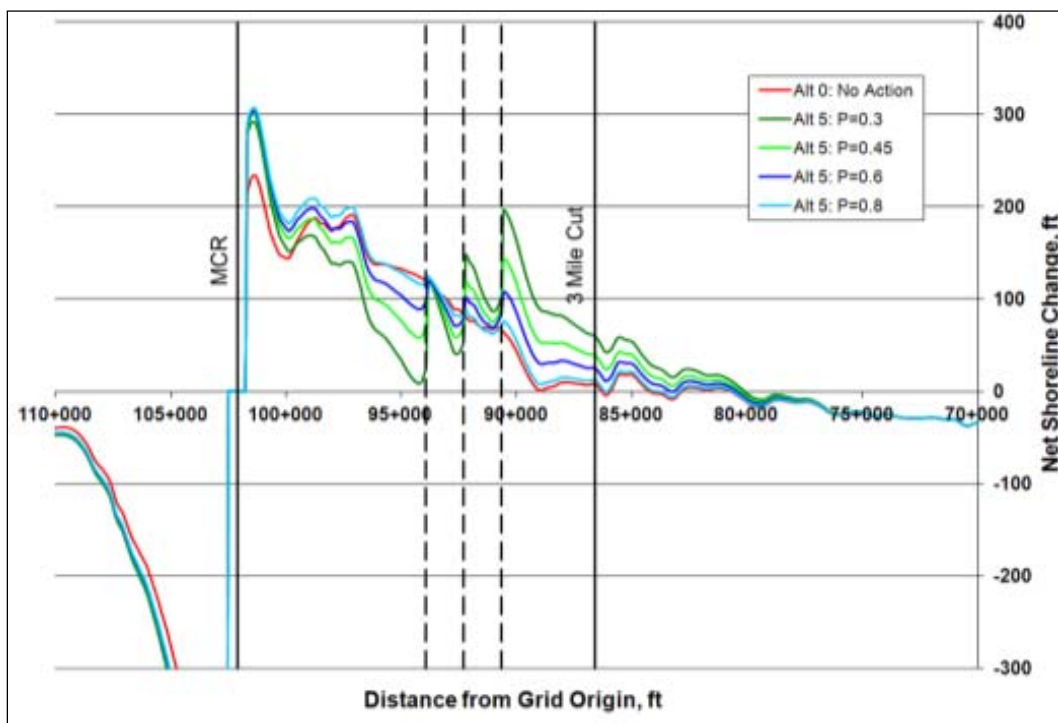
Figure A47. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 16 yr with no bypassing and a 200-ft wide beach fill.
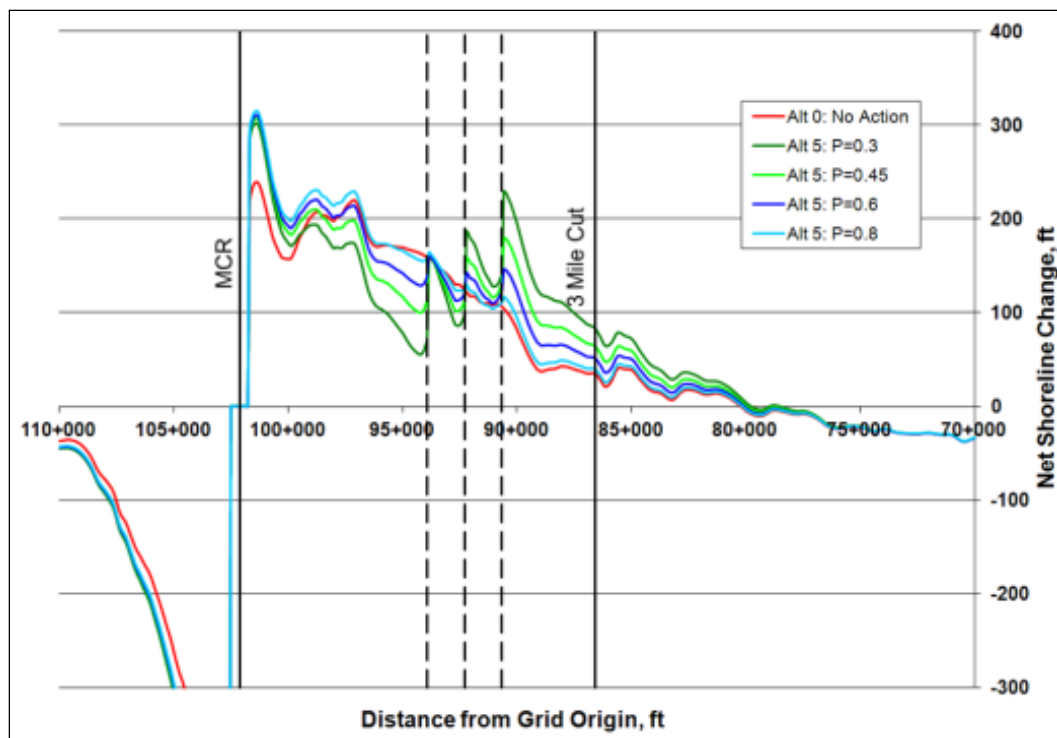
Figure A48. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 16 yr with 200,000 yd³/yr and no beach fill constructed.
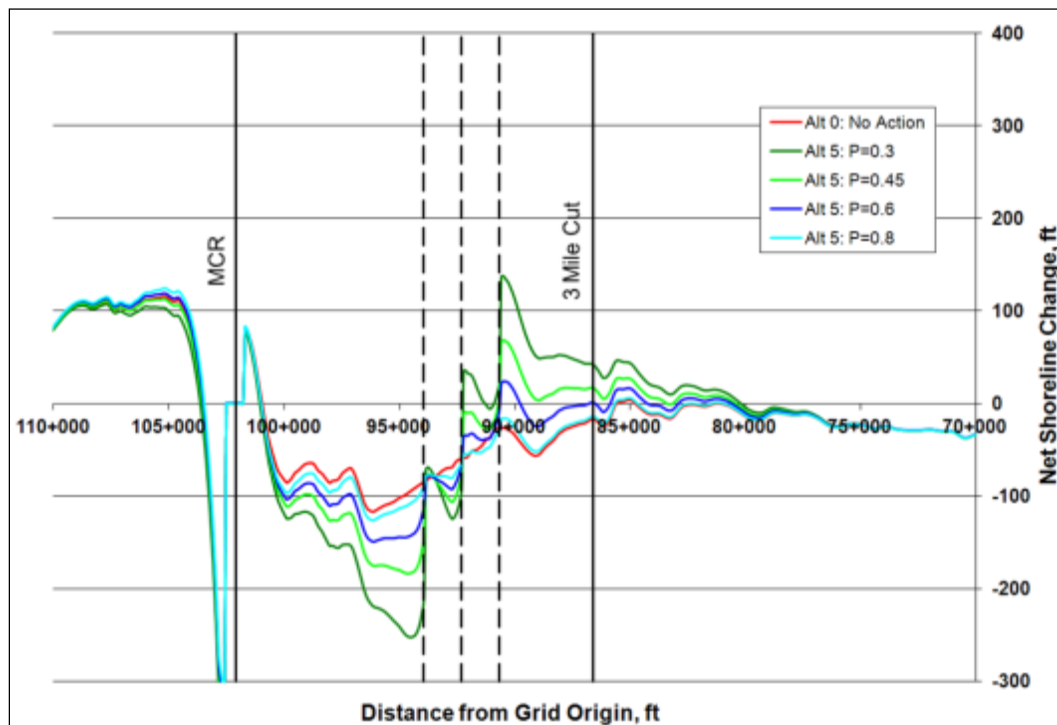
Figure A49. Simulated shoreline change between MCR and 3 Mile Cut for Alternatives 0 and 5 (all porosities) after 5 yr with 200,000 yd³/yr and a 200- ft wide beach fill.
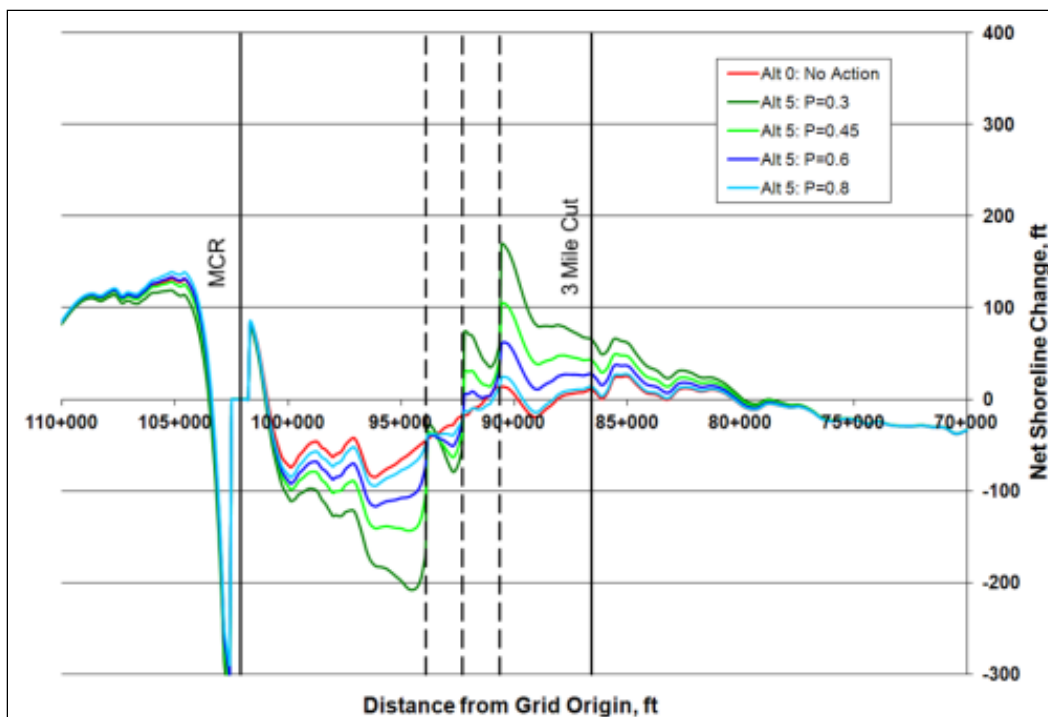
The table below summarizes all of the Alternative 5 cases discussed and provides details to compare each alternative or case to all others. The four different cases with Alternative 0 are listed first. Columns 2 and 3 state the shoreline accretion or recession at 3 Mile Cut after 5 and 16 yr. Columns 4 and 5 calculate the difference between each case and Alternative 0 with 0 yd³/yr bypassing and no beach fill constructed. Columns 6 and 7 list the shoreline change at the cell just to the north of the first groin. Columns 8 and 9 calculate the difference between each alternative and Alternative 0 without any action. Columns 10 and 11 state the shoreline change directly downdrift of the final groin at 5 yr and 16 yr. This number represents the shoreline change in the cell next to the groin. In many of these cases, the greatest erosion occurs further downdrift than this cell. Therefore, this shoreline change might not be the maximum erosion for each case. Columns 12 and 13 calculate the difference between each alternative and Alternative 0 with no action.

Table A6. Summary of MCR results.

Alternative	3 Mile Cut		Difference b/t Alt5 & Alt0 at 3 Mile Cut		Before First Groin		Difference b/t Alt5 & Alt0 before first groin		After Last Groin		Difference b/t Alt & Alt0 after last groin	
	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)
Alt0 - 0 yd³/yr Bypassing, No Beach Fill	-2	8			24	65			42	121		
Alt0 - 0 yd³/yrr Bypassing, 200 Ft Beach Fill	19	35	21	27	90	105	66	40	108	161	66	40
Alt0 - 200K yd³/yr Bypassing, No Beach Fill	-3	-17	-1	-25	9	-26	-15	-91	-32	-86	-74	-207
Alt0 - 200K yd³/yr Bypassing, 200 Ft Beach Fill	18	11	20	3	75	14	51	-51	34	-47	-8	-168
Alt5 - P=0.3, 0 yd³/yr Bypassing, No Beach Fill	15	59	17	51	86	196	62	131	14	18	-28	-103
Alt5 - P=0.3, 115Kyd³/yr Bypassing, No Beach Fill	15	49	17	41	80	161	56	96	-30	-123	-72	-244
Alt5 - P=0.3, 200K yd³/yr Bypassing, No Beach Fill	15	42	17	34	75	137	51	72	-62	-225	-104	-346
Alt5 - P=0.3, 0 yd³/yr Bypassing, 100 Ft Beach Fill	24	71	26	63	116	213	92	148	49	41	7	-80
Alt5 - P=0.3, 115K yd³/yr Bypassing, 100 Ft Beach Fill	24	61	26	53	110	178	86	113	6	-100	-36	-221
Alt5 - P=0.3, 200K yd³/yr Bypassing, 100 Ft Beach Fill	24	54	26	46	105	153	81	88	-27	-203	-69	-324
Alt5 - P=0.3, 0 yd³/yr Bypassing, 200 Ft Beach Fill	33	83	35	75	145	229	121	164	83	64	41	-57

Alternative	3 Mile Cut		Difference b/t Alt5 & Alt0 at 3 Mile Cut		Before First Groin		Difference b/t Alt5 & Alt0 before first groin		After Last Groin		Difference b/t Alt & Alt0 after last groin	
	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)
Alt5 - P=0.3, 115K yd^3/yr Bypassing, 200 Ft Beach Fill	33	73	35	65	140	194	116	129	40	-77	-2	-198
Alt5 - P=0.3, 200K yd^3/yr Bypassing, 200 Ft Beach Fill	33	66	35	58	135	169	111	104	8	-180	-34	-301
Alt5 - P=0.45, 0 yd^3/yr Bypassing, No Beach Fill	9	38	11	30	62	142	38	77	31	65	-11	-56
Alt5 - P=0.45, 0 yd^3/yr Bypassing, 200 Ft Beach Fill	28	64	30	56	125	179	101	114	97	107	55	-14
Alt5 - P=0.45, 200K yd^3/yr Bypassing, No Beach Fill	8	17	10	9	49	65	25	0	-42	-157	-84	-278
Alt5 - P=0.45, 200K yd^3/yr Bypassing, 200 Ft Beach Fill	27	43	29	35	112	102	88	37	24	-117	-18	-238
Alt5 - P=0.6, 0 yd^3/yr Bypassing, No Beach Fill	4	25	6	17	45	107	21	42	42	93	0	-28
Alt5 - P=0.6, 0 yd^3/yr Bypassing, 100 Ft Beach Fill	14	38	16	30	77	126	53	61	74	113	32	-8
Alt5 - P=0.6, 0 yd^3/yr Bypassing, 200 Ft Beach Fill	25	51	27	43	109	146	85	81	106	133	64	12
Alt5 - P=0.6, 200K yd^3/yr Bypassing, No Beach Fill	3	1	5	-7	30	21	6	-44	-31	-121	-73	-242
Alt5 - P=0.6, 200K yd^3/yr Bypassing, 100 Ft Beach Fill	13	14	15	6	63	40	39	-25	1	-101	-41	-222

Alternative	3 Mile Cut		Difference b/t Alt5 & Alt0 at 3 Mile Cut		Before First Groin		Difference b/t Alt5 & Alt0 before first groin		After Last Groin		Difference b/t Alt & Alt0 after last groin	
	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)	5 yr (ft)	16 yr (ft)
Alt5 - P=0.6, 200K yd³/yr Bypassing, 200 Ft Beach Fill	24	28	26	20	95	60	71	-5	34	-82	-8	-203
Alt5 - P=0.8, 0 yd³/yr Bypassing, No Beach Fill	0	12	2	4	27	76	3	11	50	116	8	-5
Alt5 - P=0.8, 0 yd³/yr Bypassing, 200 Ft Beach Fill	21	39	23	31	94	116	70	51	113	156	71	35
Alt5 - P=0.8, 200K yd³/yr Bypassing, No Beach Fill	-1	-14	1	-22	12	-17	-12	-82	-22	-93	-64	-214
Alt5 - P=0.8, 200K yd³/yr Bypassing, 200 Ft Beach Fill	20	14	22	6	78	24	54	-41	42	-55	0	-176

Sargent Beach shoreline modeling – additional cases

Sargent Beach construction Phase 2

Phase 2 recommends 15 breakwaters beginning just to the northeast of Mitchell's Cut. This phase includes the first 10 breakwaters from Phase 1, so only five additional breakwaters would need to be constructed. The breakwaters simulated in this phase are a constant 220 ft long and all of the gaps between the breakwaters are 330 ft. Figure A50 shows the grid and breakwaters. The analysis for this phase does not include the additional sensitivities of breakwater lengths and gap sizes like Phase 1. The breakwaters were designed at distances of 350 ft and 400 ft offshore. The simulations for this phase were run for 5 and 16 yr, but only the 5-yr case is discussed here.

Figure A50. GenCade setup for Phase 2 (15 breakwaters shown in orange).
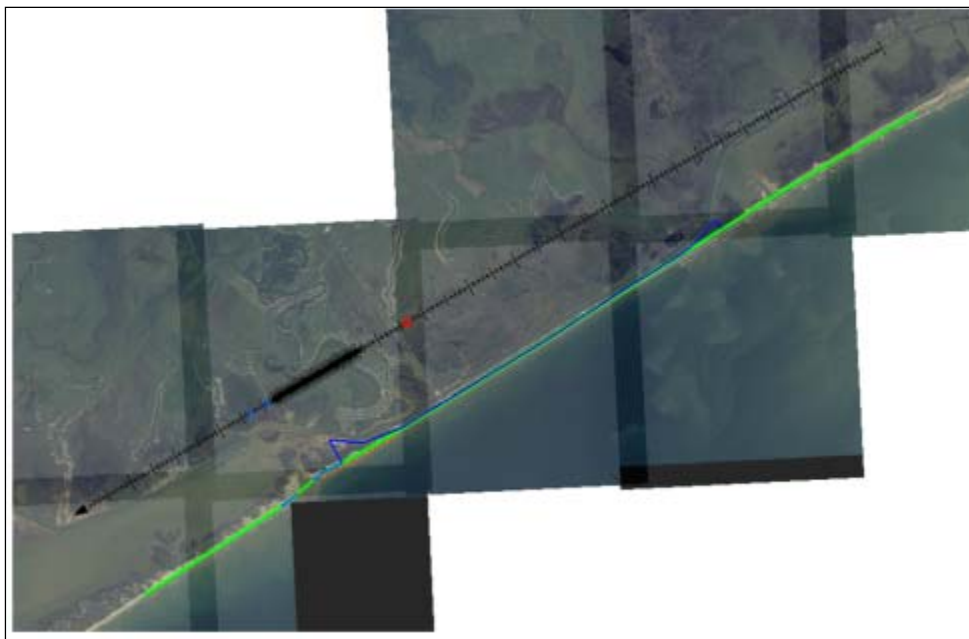
Figures A51, A52, and A53 compare the shoreline change after 5 yr in Phase 2 to Alternative 0: No Action. Figure A51 compares the shoreline change for the cases when breakwaters located 350 ft offshore are constructed with and without the 36,000 yd3/yr beach fill. Including the beach fill only affects the shoreline to the southwest of Mitchell's Cut. When the beach fill is placed, there is slightly more erosion than Alternative. 0: No Action for about 0.5 miles directly to the southwest of Mitchell's Cut. Southwest of about 71+000 ft, the case with the 36,000 yd3/yr beach fill erodes less than Alternative 0: No Action. Tombolos form behind the northernmost and southernmost

Figure A51. Shoreline change after 5 yr for breakwaters 350 ft offshore with and without beach fill.

Figure A52. Shoreline change after 5 yr for breakwaters 400 ft offshore with and without beach fill.

Figure A53. Shoreline change after 5 yr for breakwaters at 350 ft and 400 ft offshore.
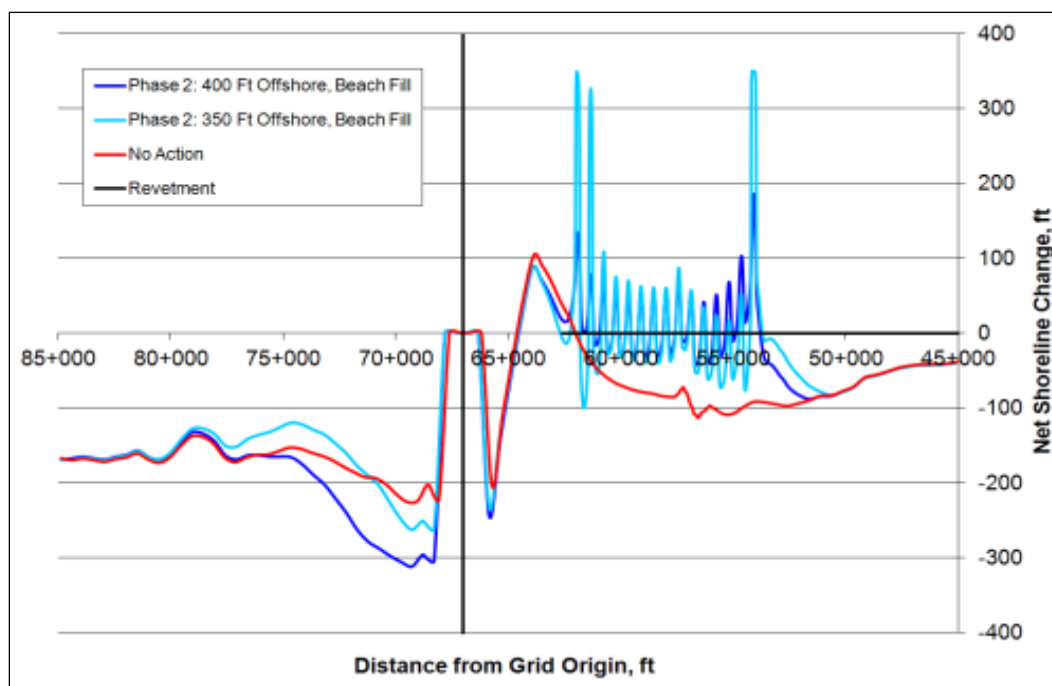
breakwaters. Overall, the shoreline accretes slightly in the region of the breakwaters. Figure A52 compares the beach fill and without beach fill cases for breakwaters located 400 ft offshore. The cases are identical to the northeast of Mitchell's Cut. Salients form behind each breakwater, and the average position of the shoreline after 5 yr is seaward of the initial shoreline. The beach fill to the south of Mitchell's Cut decreases the erosion, but the erosion is greater than Alternative 0: No Action. The shoreline change with breakwaters located at 350 ft offshore is compared to the case with breakwaters at 400 ft offshore in Figure A53.

The gross transport rates for the case with breakwaters at 400 ft offshore were very similar to gross transport rates for the case with breakwaters at 350 ft offshore, so Figure A54 shows only the gross transport rates for the case with breakwaters at 350 ft offshore. The gross transport rates at the breakwater locations decrease from about 400,000 yd³/yr to about 215,000 yd³/yr , which is similar to the effects seen in Phase 1.

Sargent Beach intermediate phase: 30 breakwaters

A set of 30 breakwaters was also set up in GenCade, which included 15 breakwaters at the northeastern end of the revetment and 15 breakwaters at the southwestern end of the revetment. Since the recommendation is to

Figure A54. Average gross transport after 5 yr for Phase 2.
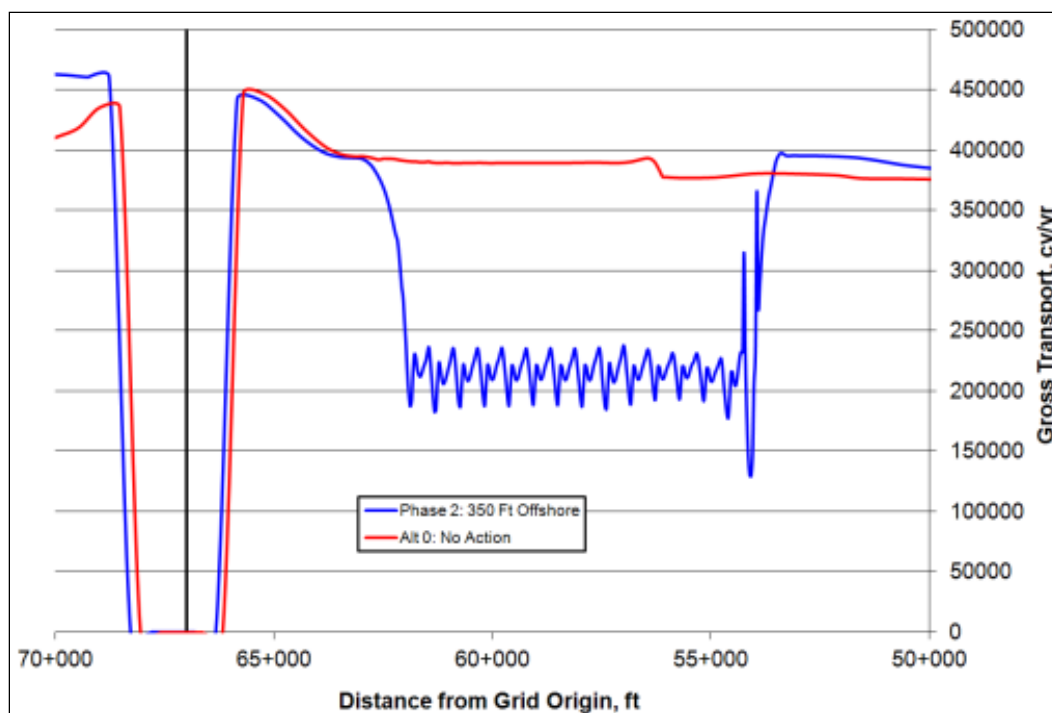
continue constructing breakwaters from south to north, it is highly unlikely that this setup of 15 breakwaters at the northeastern end of the revetment would be constructed. Nevertheless, this alternative was simulated in GenCade, and the results are shown here. Figure A55 shows the grid and the breakwaters (in orange). If this phase was to be constructed, it would occur after Phase 2, so only an additional 15 breakwaters would need to be constructed. The breakwaters simulated in this phase are a constant length of 220 ft and separated by gaps of 330 ft. Simulations with breakwaters at 350 ft and 400 ft were run for this phase.

Figure A56 compares the shoreline change for the cases with breakwaters at 350 ft offshore with and without a 36,000 yd³/yr beach fill. The two cases are identical from the beginning of the grid to Mitchell's Cut. Southwest of Mitchell's Cut, the case with the beach fill erodes slightly less than the case without the beach fill. However, 30 breakwaters will trap more sediment than Phase 1 or Phase 2, so the volume of sediment necessary for mitigation of this alternative must be greater than two previous phases.

The intermediate phase with breakwaters at 350 ft and 400 ft are compared in Figure A57. Several tombolos form after 5 yr when the breakwaters are constructed at 350 ft offshore. When the breakwaters are located at 400 ft offshore, only salients form. Both cases successfully advance the

Figure A55. GenCade setup for the intermediate phase (30 breakwaters shown in orange).
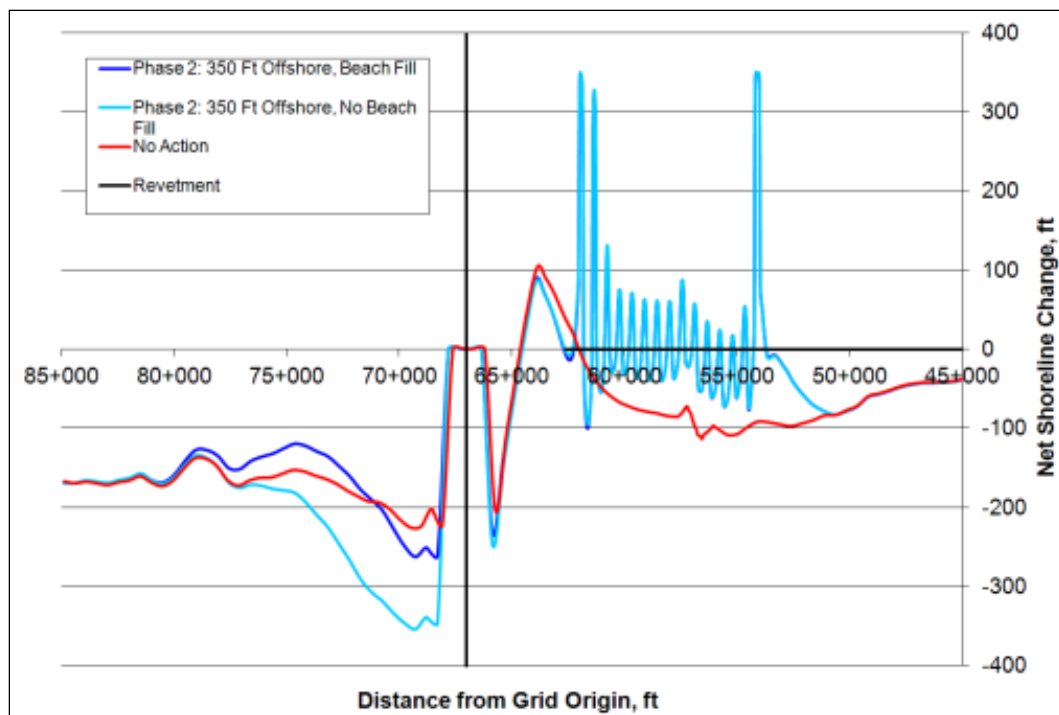

Figure A56. Shoreline change after 5 yr for breakwaters 350 ft offshore with and without beach fill.

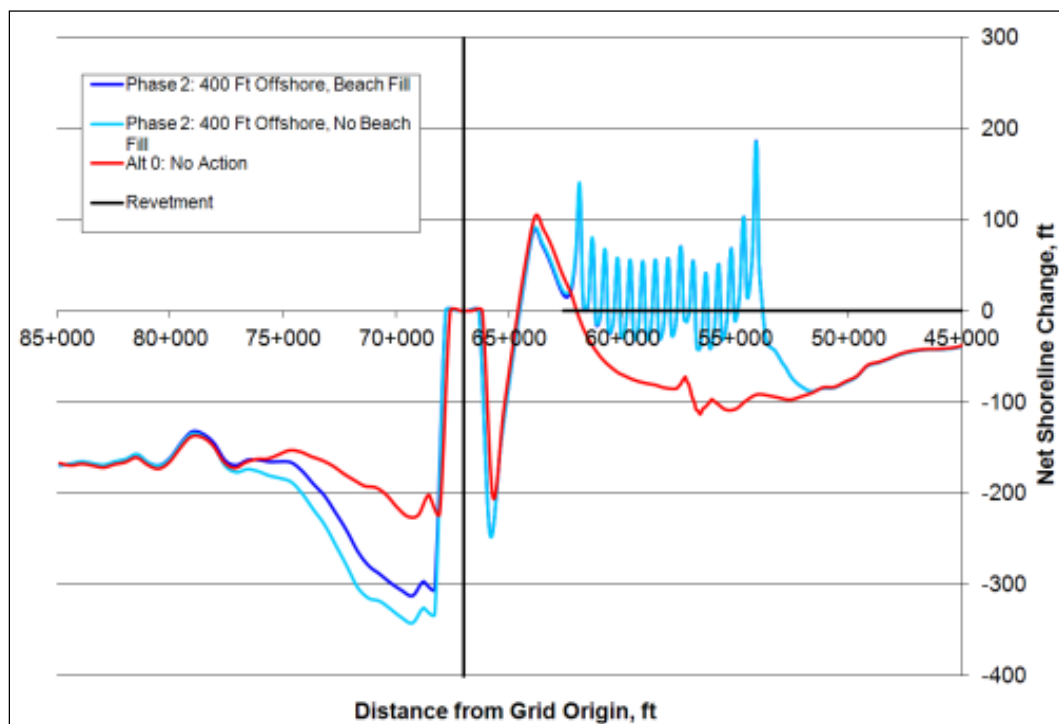Figure A57. Shoreline change after 5 yr for breakwaters at 350 ft and 400 ft offshore.
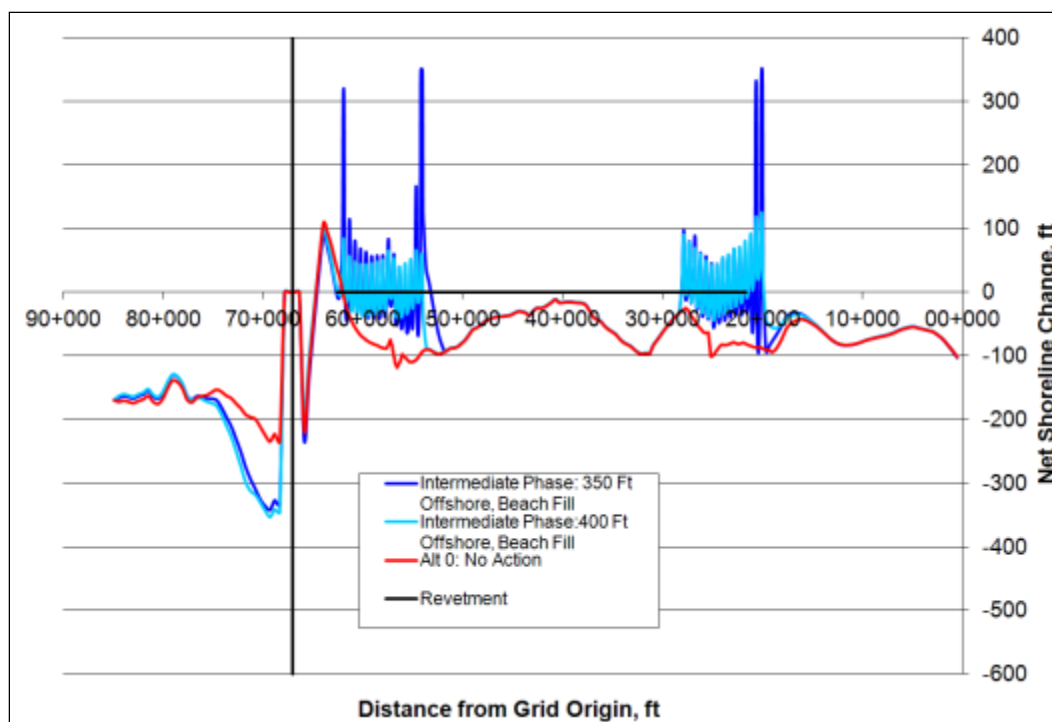
shoreline from Alternative 0: No Action behind the breakwaters. However, the shoreline erosion between the two breakwaters sections is identical to the case without any action. The shoreline erodes to the revetment between the two sections of breakwaters, so the intermediate phase does not provide any protection in this region. Figures A58 and A59 show the shoreline change at each breakwater section.

Figure A60 compares the gross transport rates of the intermediate phase to Alternative 0: No Action. The gross transport at the northern breakwaters decreases about 50 % between Alternative 0: No Action and the intermediate phase. The gross transport between the breakwaters for the intermediate phase ranges from about 325,000 yd³/yr to 450,000 yd³/yr. Just northeast of the first breakwater, the gross transport rate of the intermediate phase is greater than Alternative 0: No Action. The gross transport decreases from about 400,000 yd³/yr for Alt 0: No Action to about 250,000 yd³/yr for the intermediate phase at the second breakwater section. Gross transport south of Mitchell's Cut increases to about 500,000 yd³/yr for the intermediate phase.

Figure A58. Shoreline change after 5 yr for northern breakwaters at 350 ft and 400 ft offshore.
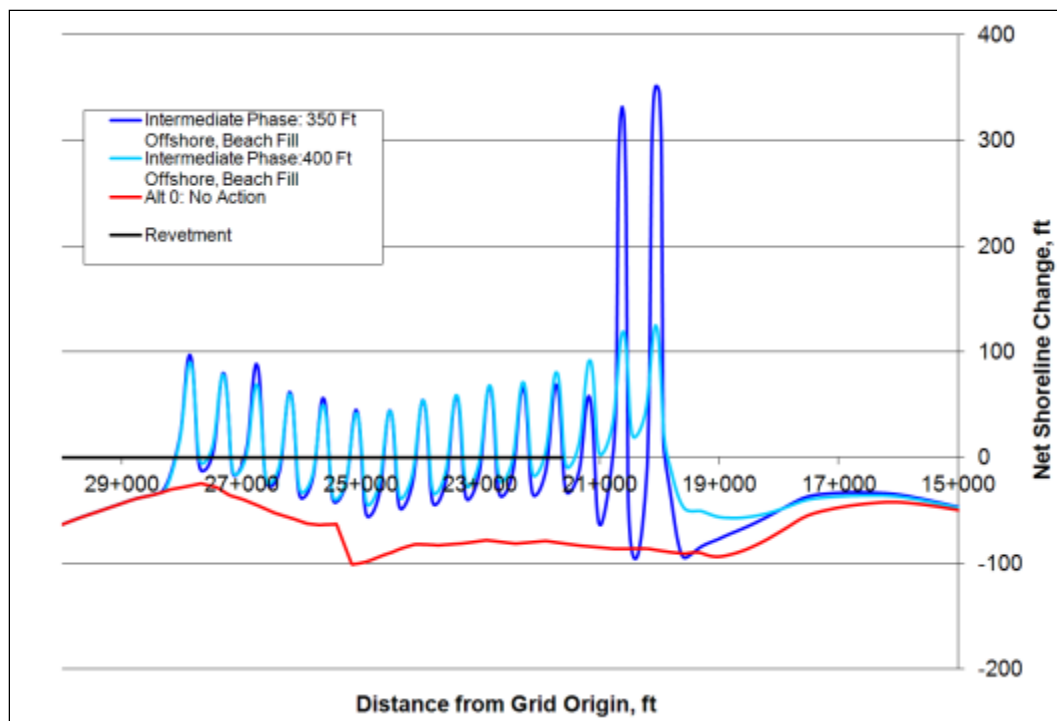
Figure A59. Shoreline change after 5 yr for southern breakwaters at 350 ft and 400 ft offshore.
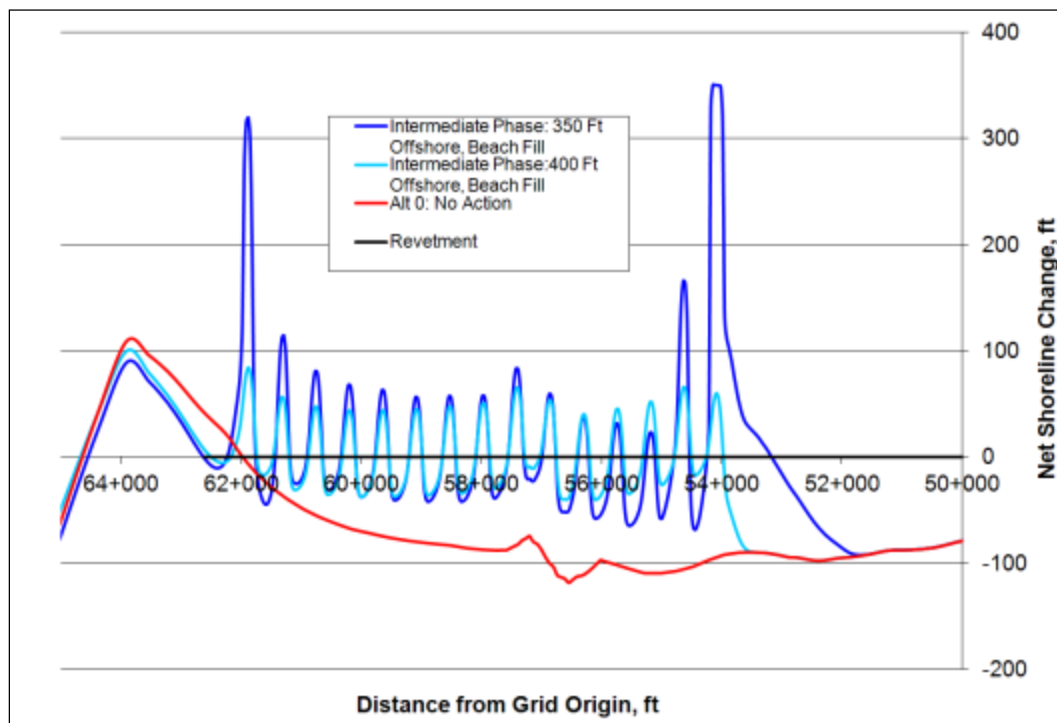
Figure A60. Average gross transport after 5 yr for intermediate phase.
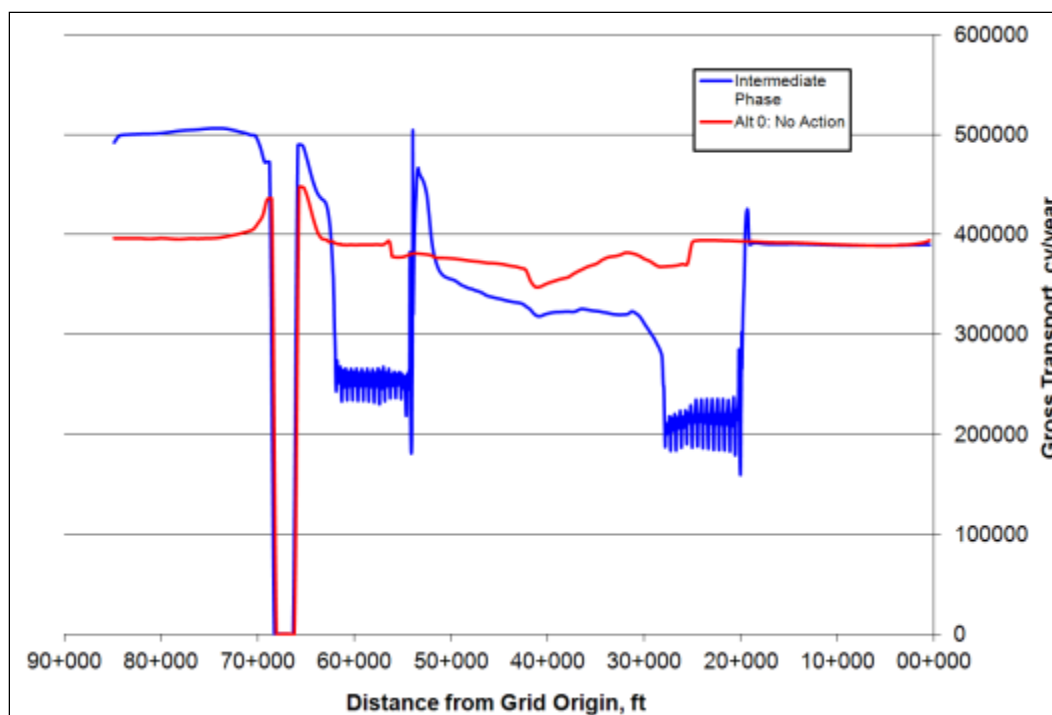
Sargent Beach final phase: 81 breakwaters

This phase requires 81 breakwaters extending from the very north end of the revetment nearly to Mitchell's Cut. Based on the 220-ft breakwater length and 330-ft gap length, the greatest number of breakwaters necessary to protect Sargent Beach is 81. Figure A61 shows the grid setup and all of the breakwaters necessary for the Final Phase. The first breakwater is located just northeast of the revetment, and the last breakwater is in the same location as Phases 1 and 2. If the final phase occurred after Phase 2 without any additional construction, 66 additional breakwaters would be necessary.

Shoreline change for all cases is shown in Figures A62-A64. Figure A62 compares shoreline change for breakwaters located 350 ft offshore with and without a 36,000 yd³/yr beach fill southwest of Mitchell's Cut. Figure A63 compares the with beach fill and without beach fill cases when breakwaters are located at 400 ft from shore. Breakwaters at 350 and 400 ft offshore are compared when the beach fill is present in Figure A64. After 5 yr, the shoreline change for the Final Phase for breakwaters at 350 ft and 400 ft offshore is very similar. The only major difference between the cases occurs just to the northeast of the first breakwater. About 80 ft of sediment builds up to the northeast of the first breakwater when the breakwaters are located 350 ft from shore. While there is slight accretion to the north of the first

breakwater with breakwaters 400 ft offshore, it is not as significant as the other case. Additionally, large erosion will occur just to the north of the revetment in all cases.

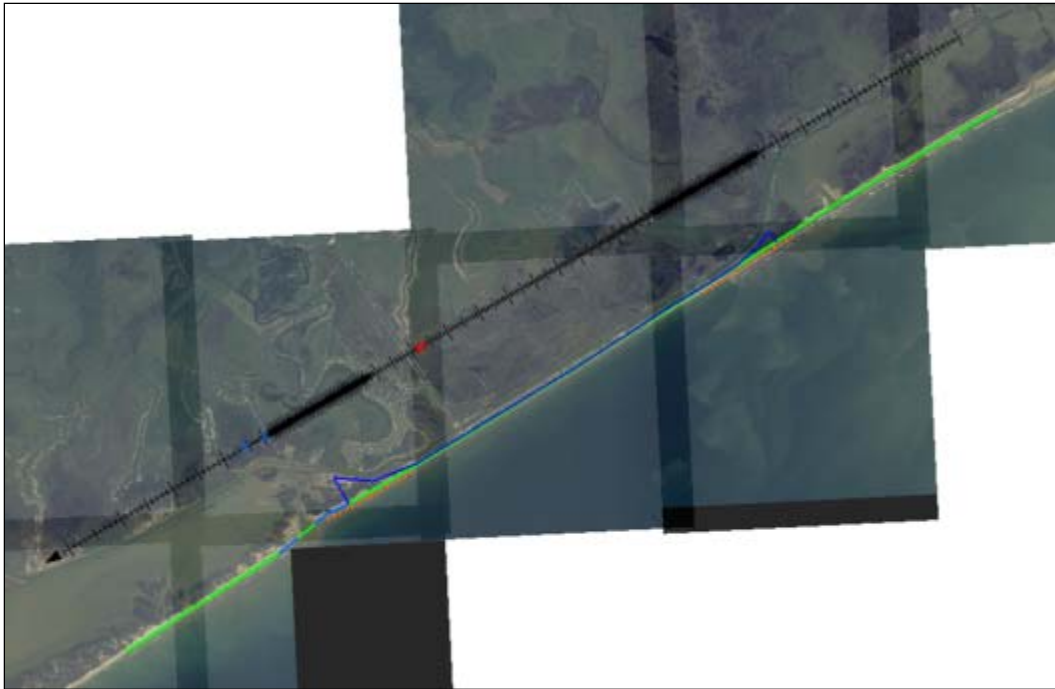Figure A61. GenCade setup for final phase (all breakwaters shown in orange).
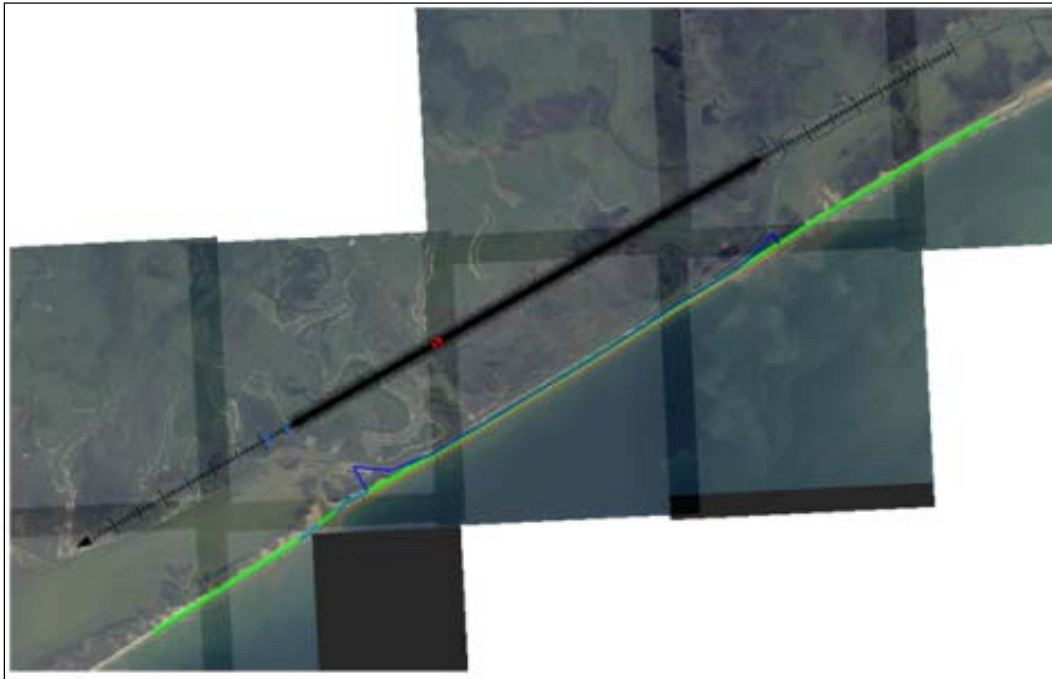
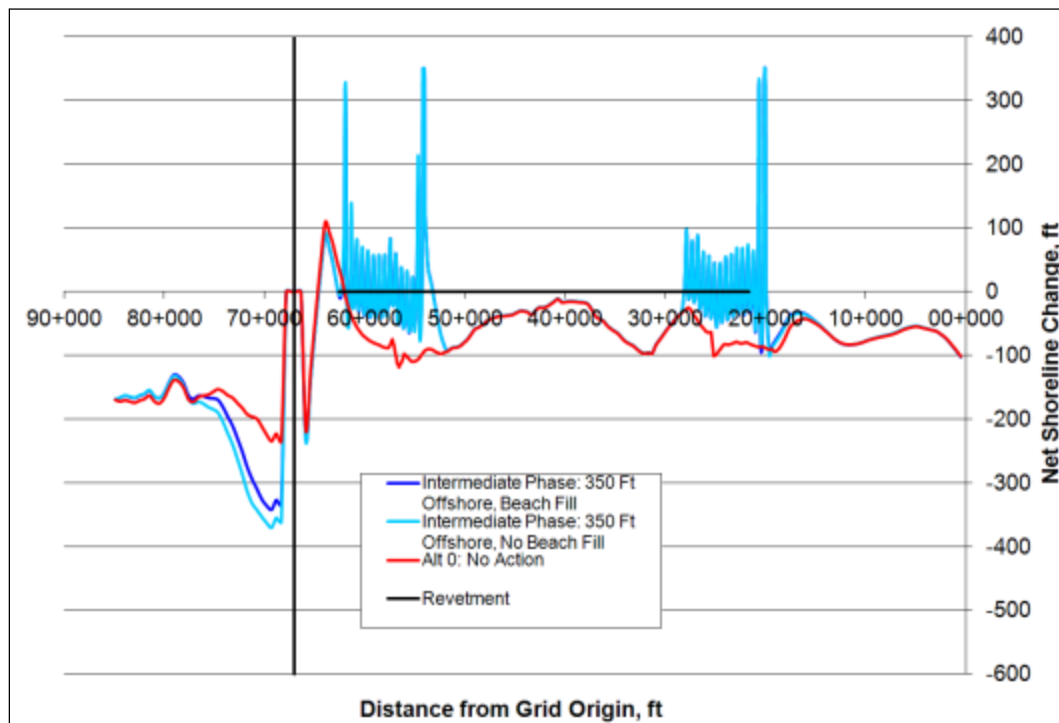Figure A62. Shoreline change after 5 yr for breakwaters at 350 ft offshore with and without beach fill.
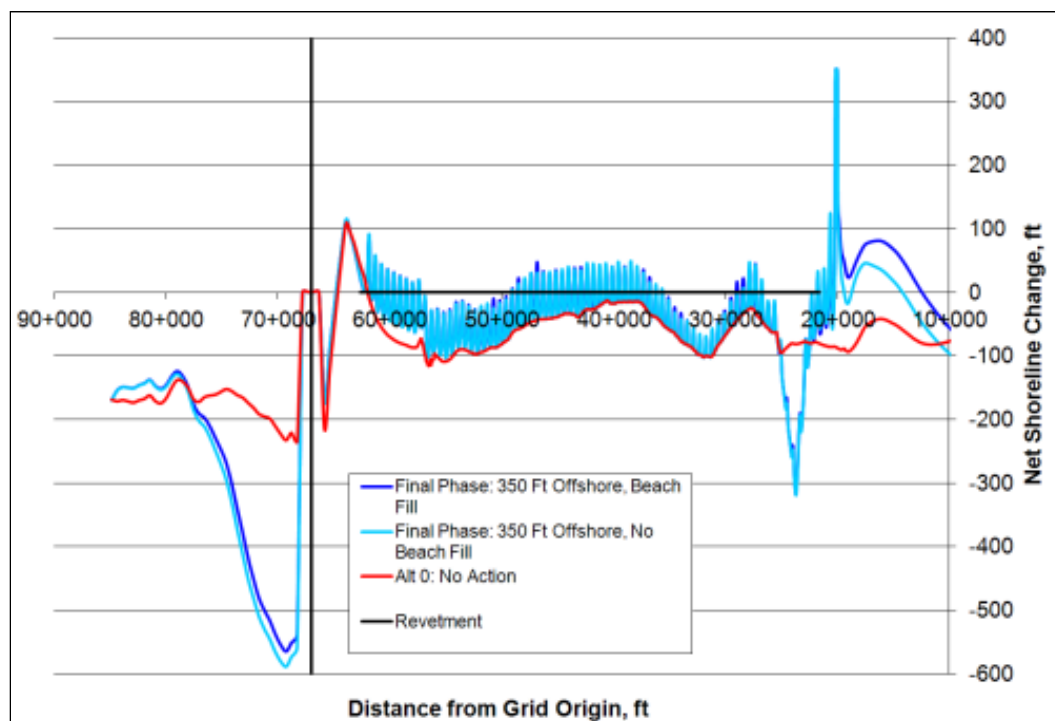
Figure A63. Shoreline change after 5 yr for breakwaters at 400 ft offshore with and without beach fill.
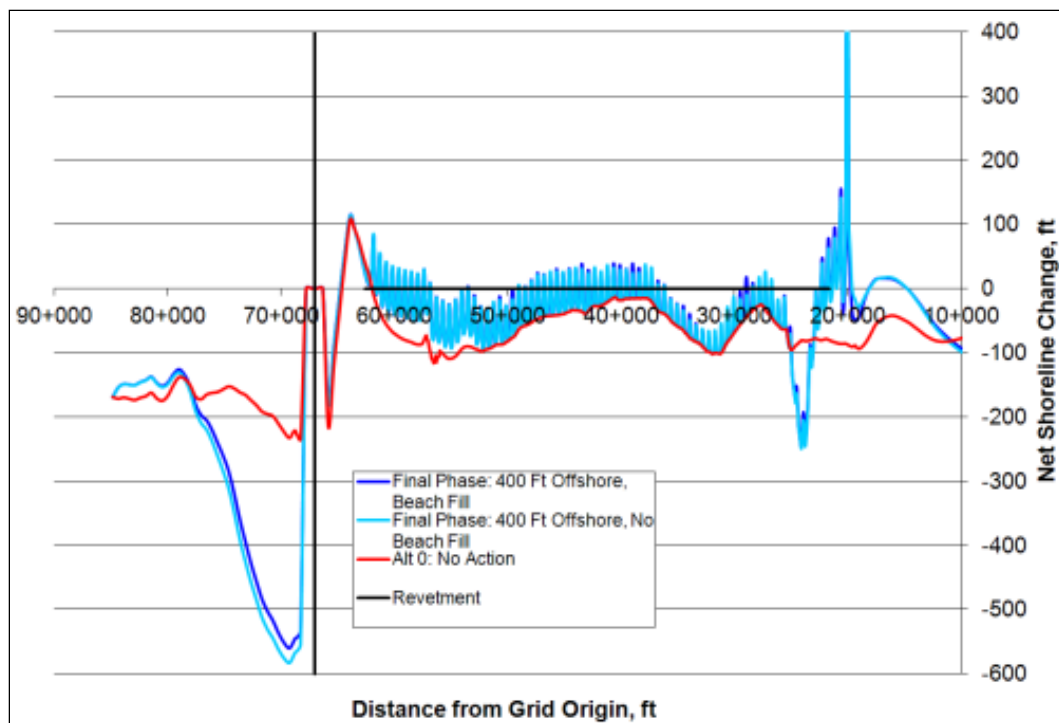
Figure A64. Shoreline change after 5 yr for breakwaters at 350 and 400 ft offshore.
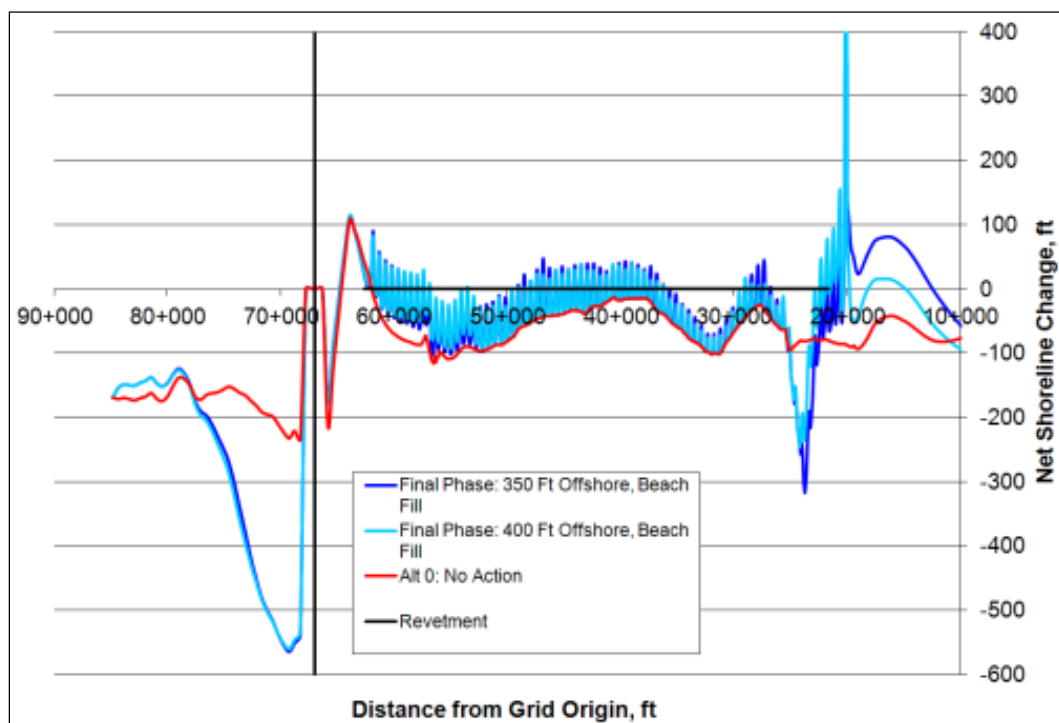
Figure A65 shows the gross transport averaged over 5 yr for the final phase. The gross transport is identical to Alternative 0: No Action until the first breakwater. Then the gross transport decreases to about 200,000 yd³/yr . The gross transport increases moving south. At the last breakwater, the gross transport is about 440,000 yd³/yr.

Figure A65. Average gross transport over 5 yr for final phase.
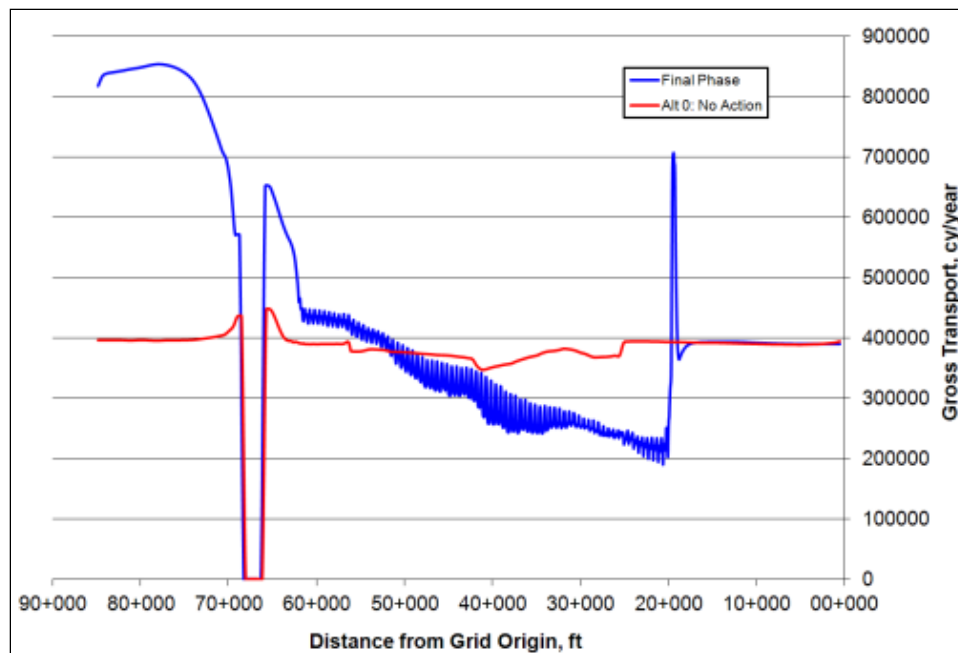
Sargent Beach Phase 1: Mitchell's Cut removed from GenCade grid

As stated previously, if the breakwaters were located in the same place as the breakwaters in the CMS, the cell size required for the breakwaters would be too small to represent the shoreline change at Mitchell's Cut. Therefore, the breakwaters were moved about 0.5 miles to the northeast, so that the cell size in the vicinity of Mitchell's Cut was large enough to predict reasonable shoreline change. Because the breakwaters were moved further to the north in all of the previous sections, it was necessary to create a grid with breakwaters in the same location as the CMS. Mitchell's Cut was removed in this case. Figures A66 and A67 describe the shoreline change and average gross transport for the case where breakwaters were located 400 ft offshore. Even though the inlet was removed from GenCade, the initial shoreline was not altered. This is the reason that Alternative 0: No Action, No Inlet shows a large advance in shoreline change to the east of Mitchell's Cut and significant retreat to the west. Tombolos have formed behind more of the breakwaters nearest to the inlet, but the overall trend of

the shoreline in the region of the breakwaters is a slight advance. The gross transport averaged over 5 yr is very similar to the GenCade results already discussed. The gross transport at the breakwaters decreases from about 400,000 yd³/yr to about 250,000 yd³/yr when breakwaters are present. Instead of gross transport decreasing to 0.0 yd³/yr at the inlet, the gross transport rate remains around 400,000 yd³/yr when the inlet was removed.

Figure A66. Shoreline change for Phase 1: No inlet, 10 breakwaters, no beach fill.
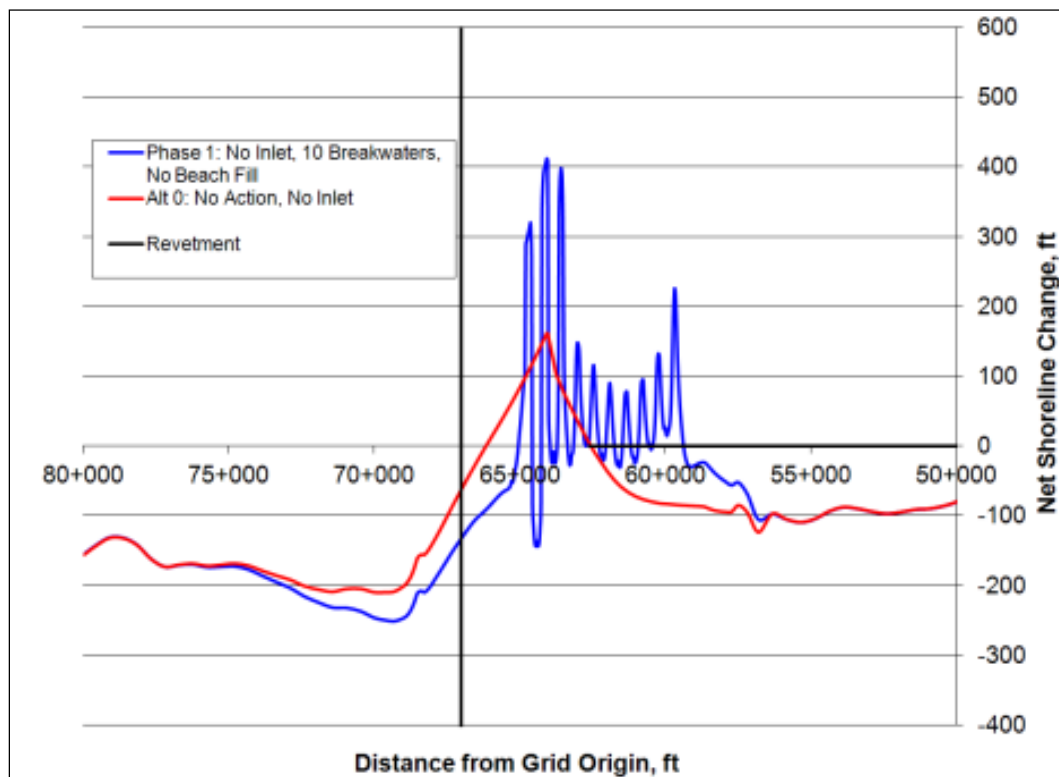
Figure A67. Average gross transport over 5 yr for Phase 1: No inlet, 10 breakwaters, no beach fill.
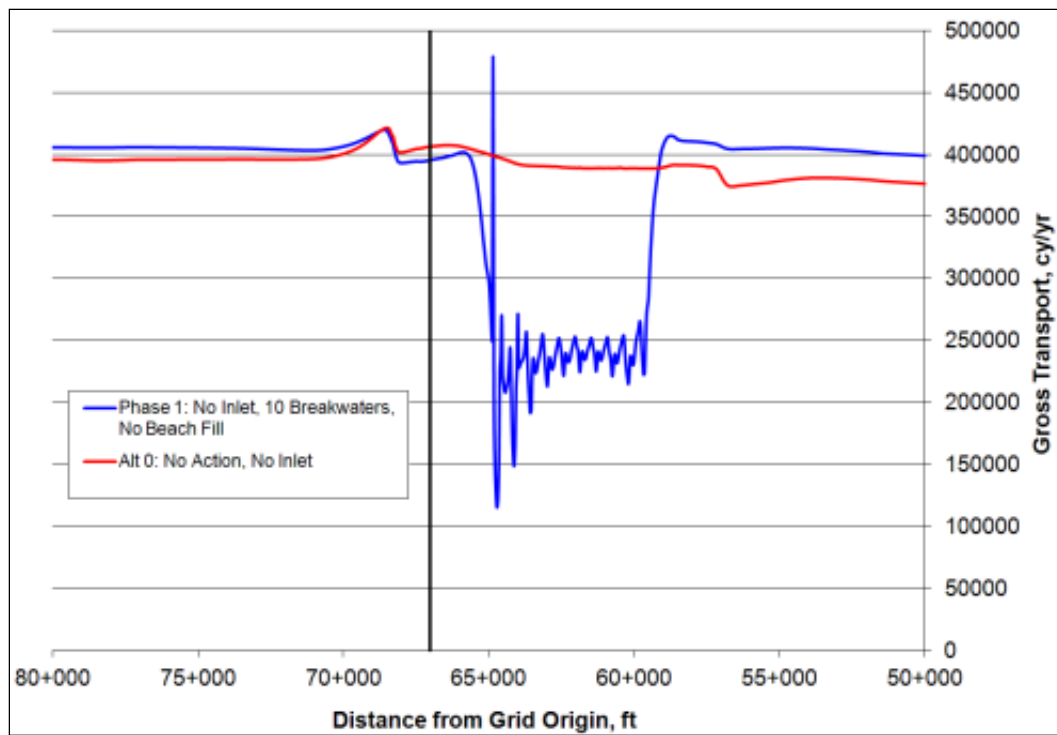
Appendix B: Numerical Modeling of Circulation, Waves, and Morphology Change

A comprehensive modeling suite, the Coastal Modeling System (CMS http://cirp.usace.army.mil/products/index.html), which calculates waves, flow, sediment transport, and morphology change, was applied in this study to evaluate with-project changes to waves and currents at the entrance to Mitchell's Cut.

Data sources

Bathymetry

Table B1 summarizes the bathymetry data sources used for this work. A bathymetric survey of Sargent Beach was performed in September 2011 as part of Phase 2 (Figure B1). The other surveys are not shown in the figure.

Winds

Wind on the present project grid was specified by measurements at the Port O'Connor gauge. Winds are also available at the Brazos River Floodgates from the USGS; however, the Port O'Connor gauge was used since its open water location gives a better indication (generally higher than land-based locations) of the winds affecting water movement, especially in East Matagorda Bay. Winds from Port Lavaca are not available during this time period. The nearest official National Weather Service reporting station is Bay City, Texas.

Table B1.Bathymetry data sources.

Date	Description
Sept 2011	Sargent Beach, Full profile (USACE ERDC)
Feb. 2010	Caney Creek, Mitchell's Cut (Coastal Tech survey)
2010	Post-Ike LIDAR (USACE)
2006	Matagorda County 2006 LIDAR dataset
2004	Lin, Kraus dataset
2001	Coastal Database 2001 (NOAA)

Figure B1. Extent of September 2011 Phase 2 survey.
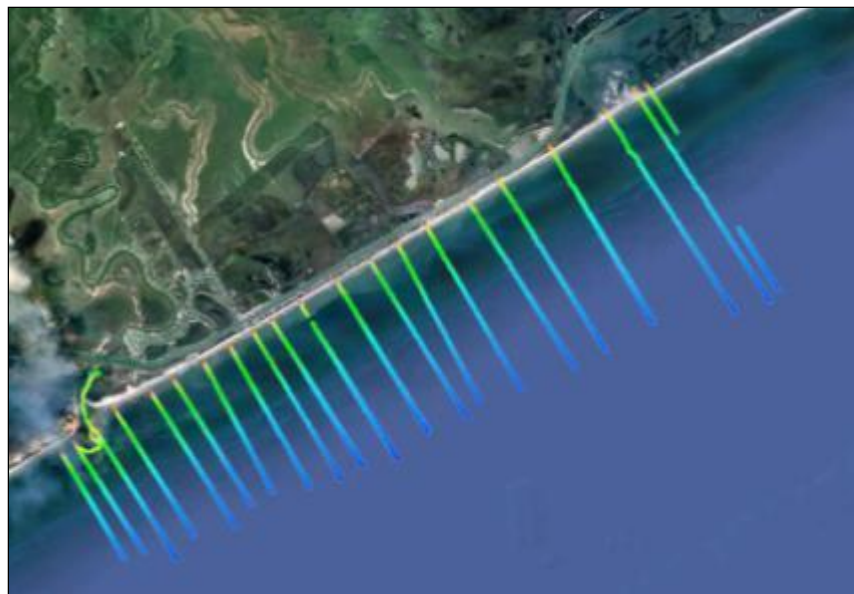
Water levels

Gulf of Mexico forcing was specified by data available from the National Oceanic and Atmospheric Administration (NOAA) water level gauge at the Galveston Pleasure (Flagship) Pier. This gauge includes water level as influenced by wind blowing over the gulf in its vicinity. A water level gauge was installed at the FM 457 Bridge in the Gulf Intracoastal Waterway (Figure B2) on September 22, 2011. Also available are water levels from the Brazos River Flood gates and the locks at the Colorado River – GIWW intersection. Both of these locations have their data reported and recorded by the USGS (http://waterdata.usgs.gov). These gauges are supported by the Corps of Engineers and report their data according to the current Corps Standard Datum of MLT (Mean Low Tide) for this area.

A value of 0.53 m was used to transform these data to the local mean sea level (MSL) datum used in the model (USACE 1989). For this study period, water levels from the east lock of the Colorado River were used. Brazos water levels from the west lock and west side of the river are not available for this time period. Therefore, water levels through the Brazos River west floodgate were simulated. As others have noted (Sanchez and Parchure 2001), the Brazos River floodgates are generally open, as confirmed by review of the eastern side data. However, the actual state of the western lock gates is not documented during this time period.

Figure B2. Water level gauge location.
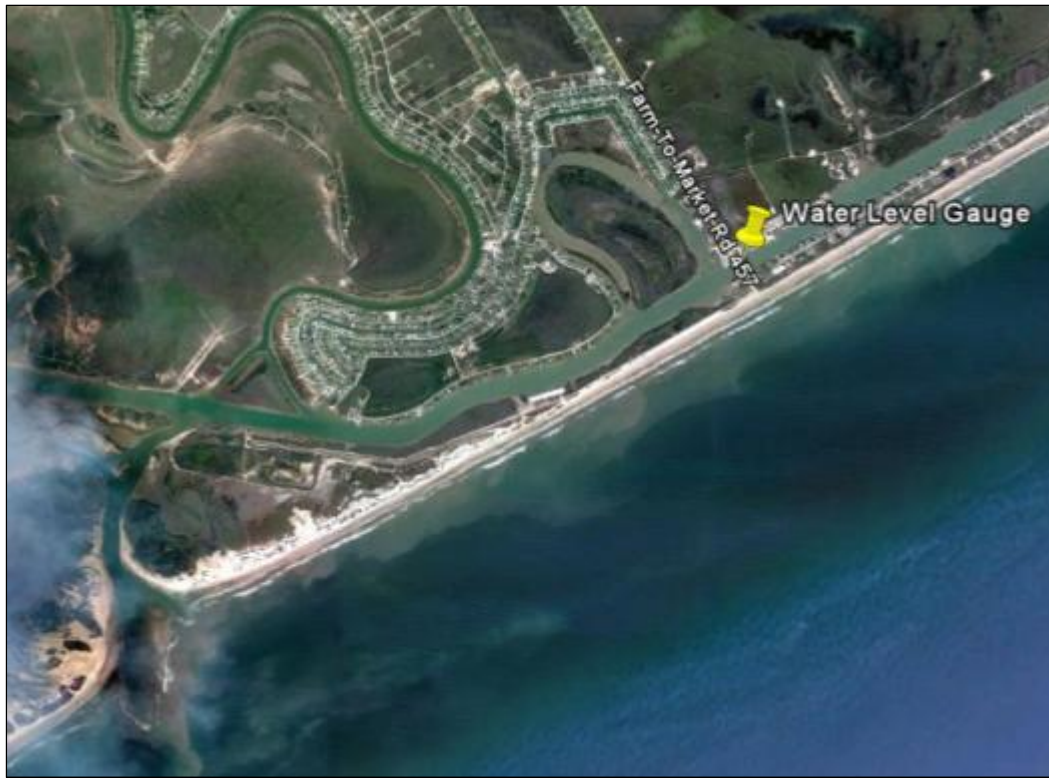
River discharges

Discharge (rate of streamflow) for the San Bernard River was taken from the USGS gauge 40 miles upstream near Boling, Texas. As stated on the Galveston District website, river flows take over a day to travel downstream from this measurement location. NOAA reports (http://water.weather.gov/ahps2/hydrograph.php?wfo=hgx&gage=swyt2) current gauge levels and discharge from the San Bernard River near Sweeny, Texas (swyt2), approximately 20 miles upstream, but historical data are not available. As per Kraus et al. (2006), the discharge for Caney Creek is taken to be 10 % of the San Bernard River value. As per Sanchez and Parchure (2001), Brazos River discharge was taken from the gauge near Rosharon, Texas.

Waves

All available data from NDBC wave buoy 42019 were obtained. The wave rose computed in Phase 1 was updated (Figure B3). Compared to the original Phase 1 wave rose, these data show a greater occurrence of waves from the South – South East sector. Also, a two-dimensional plot of combined wave height and period was created (Figure B4). Waves recorded during the validation and calibration periods were used as well as typical

summer condition waves from June to July 2008. Also, specific wave conditions were run to observe beach response to waves that create either northerly or southerly longshore currents.

Figure B3. Wave rose plot including NDBC 42019 Data from 1995 to 2011.
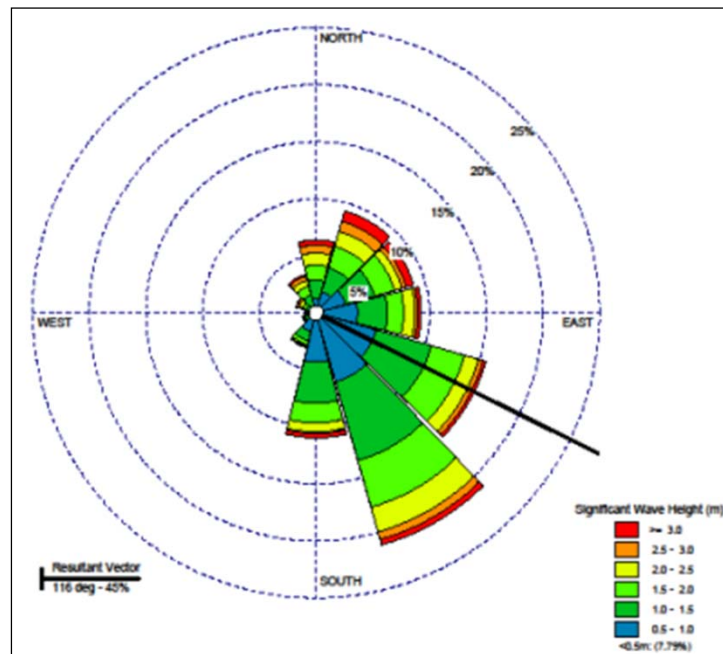
Figure B4. Joint plot of NDBC Buoy 42019 significant wave height and period.
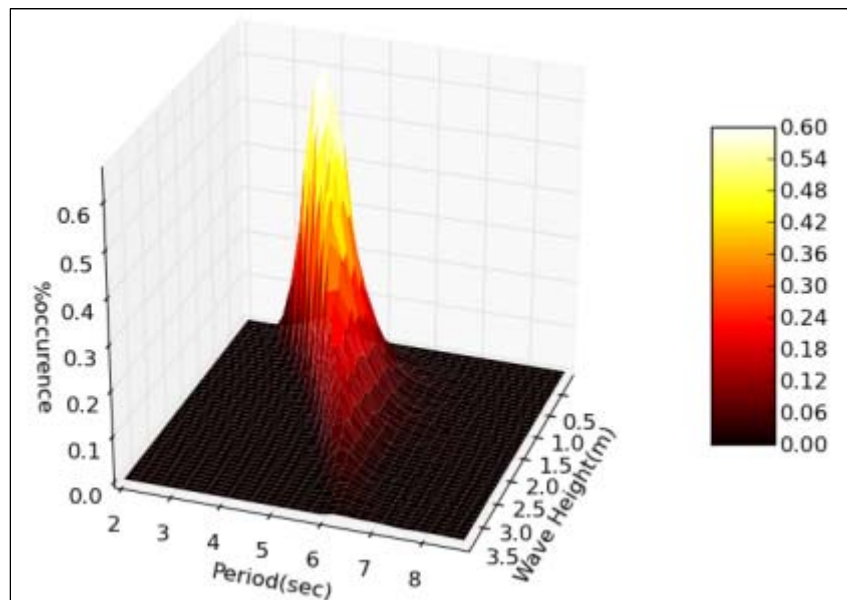
Modeling approach and grid development

In Texas bays, wind-driven water levels during seasonal highs and lows in the Gulf of Mexico can exceed changes in water level generated by astronomical tides. A large grid spanning the nearly 62 km distance between the Colorado River locks to the Brazos River floodgates was constructed to account for wind fetch across the bays and to include the available water level measurement locations. This telescoping cartesian grid consisting of more than 282,000 cells was run for hydrodynamic flow conditions only without waves or sediment transport (Figure B5).

Figure B5. Nested grid location within overall CMS grid.
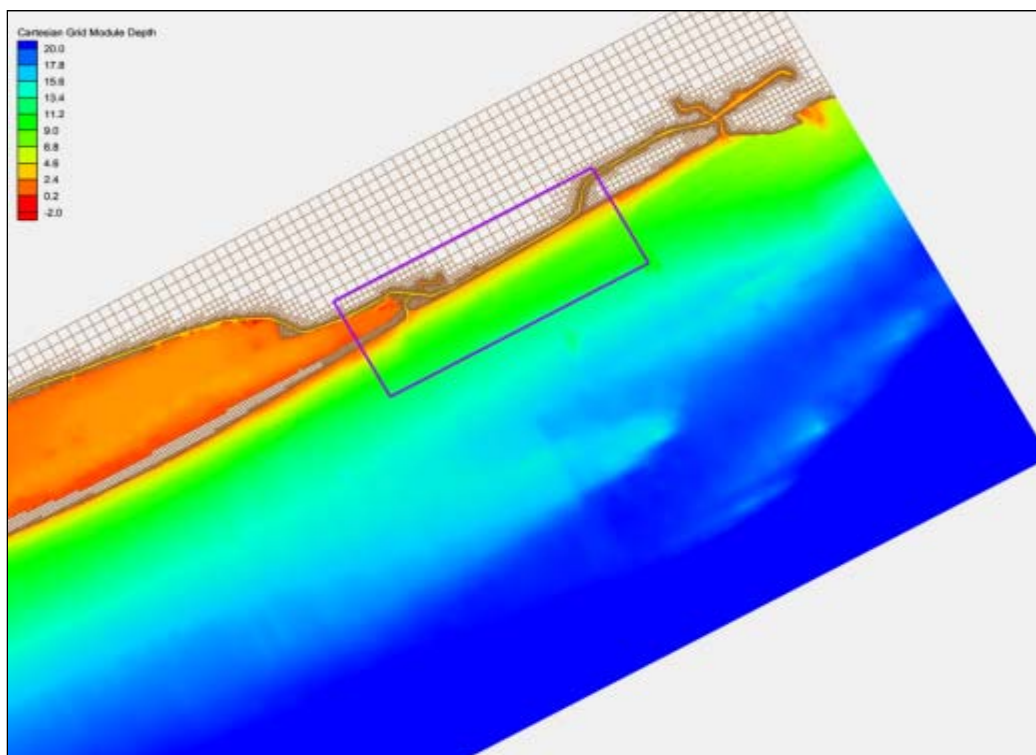
To properly account for nearshore currents, the highest grid resolution was less than 10 m in the nearshore wave breaking zone. A nested CMS-Flow grid (Figure B6) was constructed that uses water level forcing from the larger grid solution. The nested grid has more than 142,000 cells with cell resolution ranging from 100 to 3,125 m. A uniform CMS-Wave grid was also constructed for this nested extent (Figure B7). The nested telescoping grid was run with cross-shore boundary conditions enabled in CMS to more accurately model alongshore sediment transport conditions near the boundaries. To improve wave-induced sediment transport, the nested CMS grid was run with the surface roller model enabled.

Figure B6. Nested CMS flow grid.
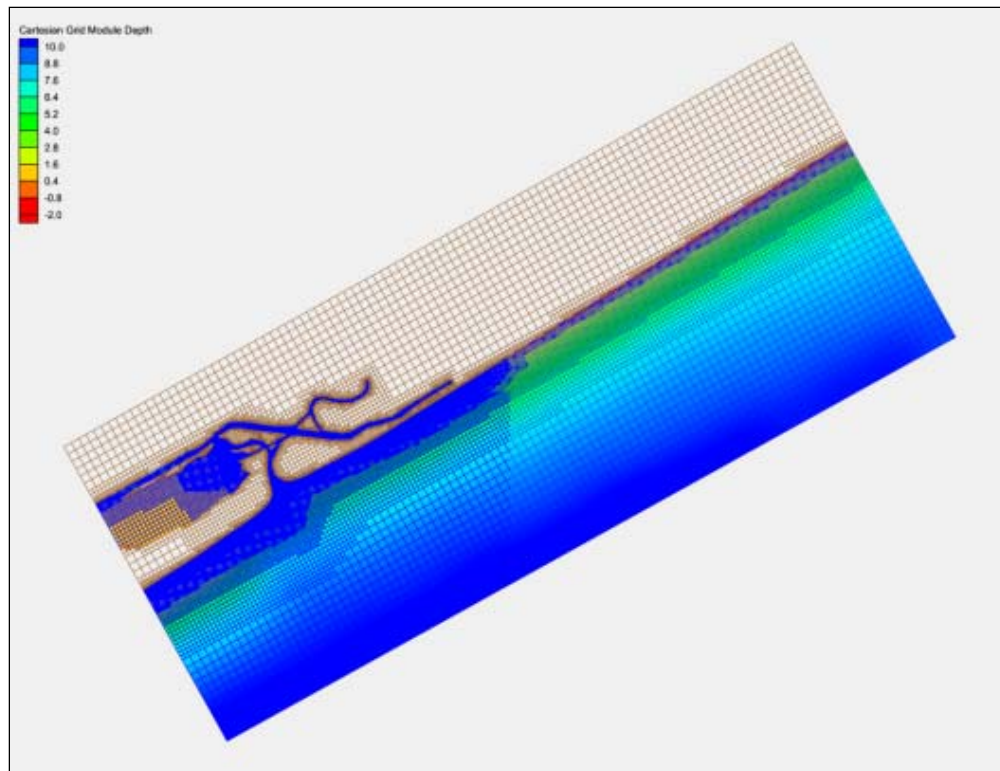
Figure B7. CMS wave grid – nested extent.
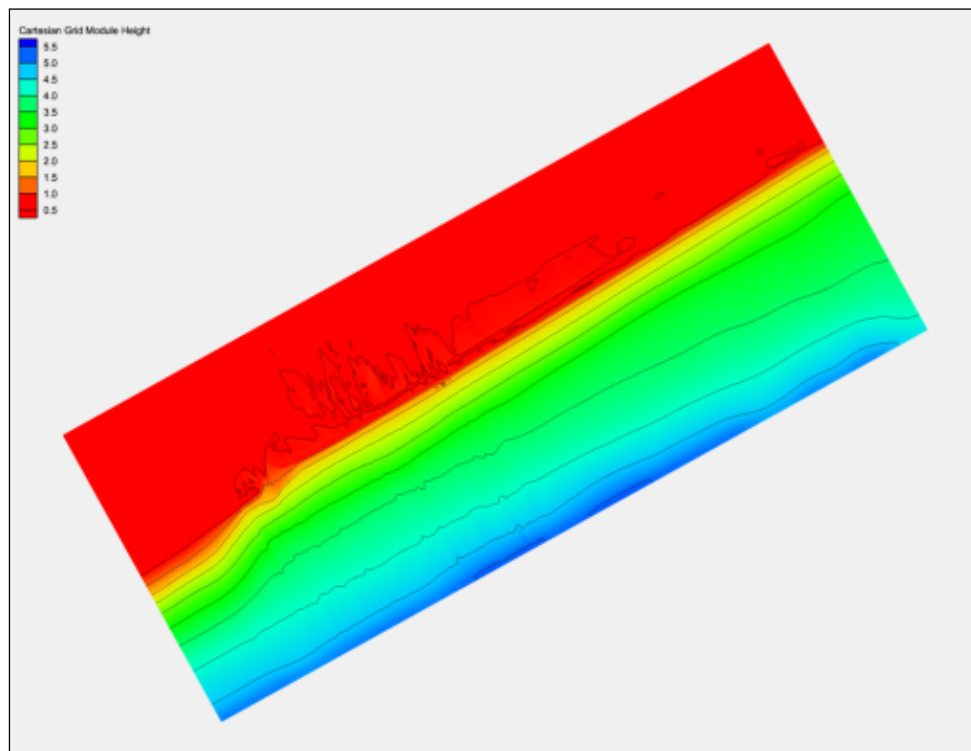
Calibration

The CMS was calibrated against data obtained from the installed tide gauge for the time period from September 23-30, 2011. Incident forcing is shown in Figure B8. The Colorado River East Lock values were low-pass filtered to remove spikes. Note the Colorado River East Locks have significantly lower values when the Colorado River is above 0.4 m MSL, supporting the hypothesis that the locks were closed at this time. The Brazos East River values are shown for comparison purposes only; they were not used for model forcing. The model was run with the west flood gate open, including inflow from the Rosharon gauge. Figure B9 shows the wind forcing from Port O'Connor for the calibration period. Figure B10 shows the comparison between measured results and computed values for the calibration period.

Computed flow patterns compared with aerial photography

To obtain a general sense if the model reproduces the relevant hydro-dynamics of the system, it is possible to compare with some available aerial photography. One instance is a San Bernard River high water event photographed on February 26, 2012 (Friendsofsanbernard.org). As part of the

Figure B8. Calibration incident forcing – Colorado River east lock used, Colorado River and measured values shown for comparison.
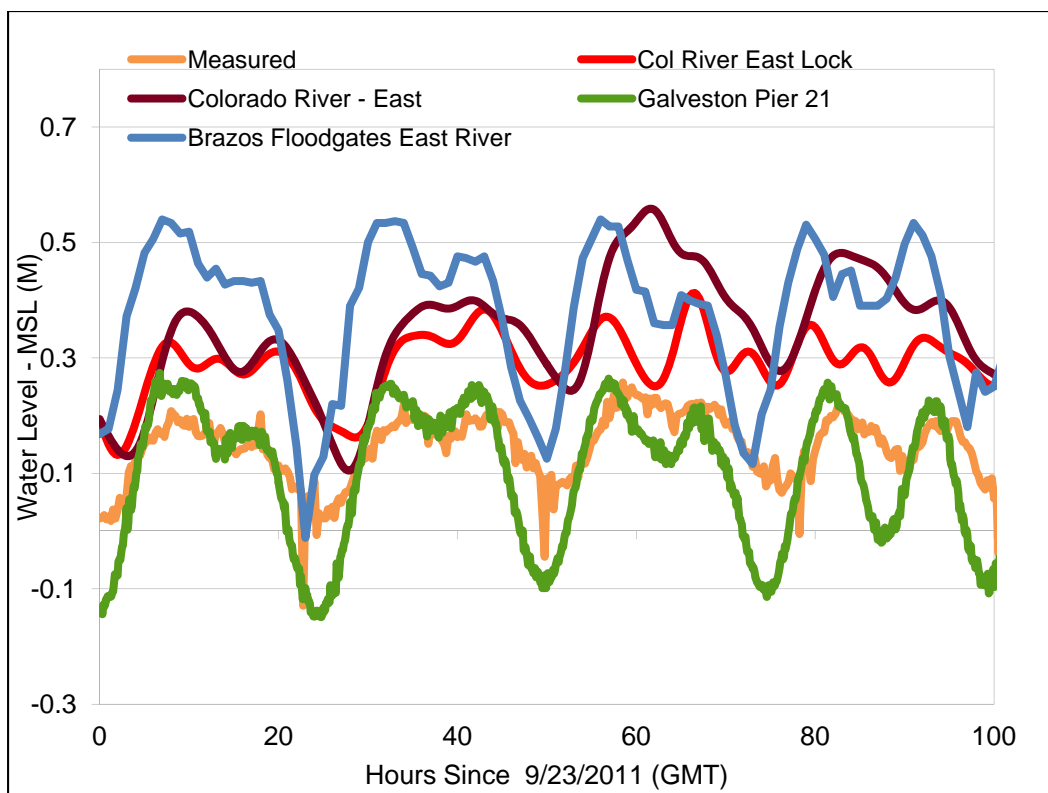
Figure B9. Calibration wind forcing from Port O'Connor.
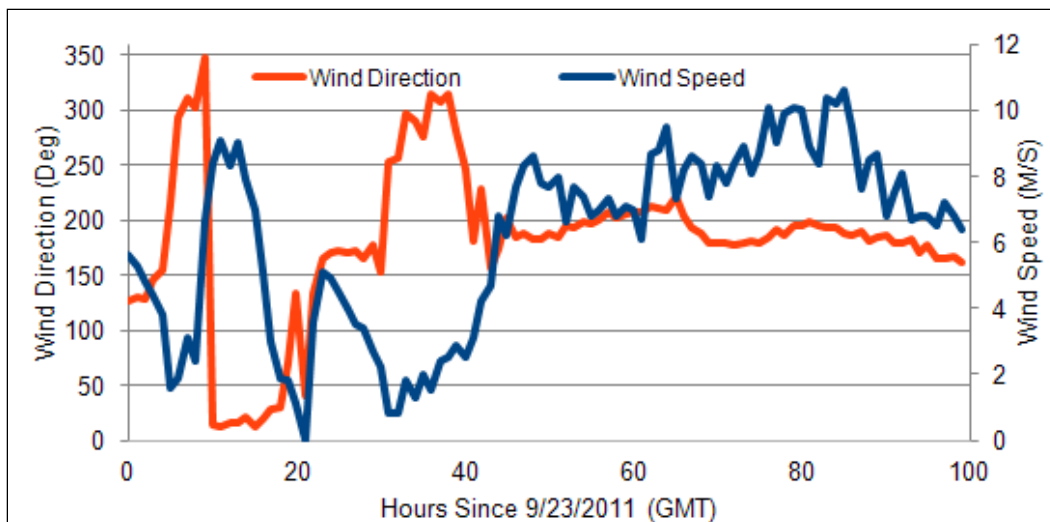
Figure B10. CMS-Flow water level calibration period comparison to measurements.
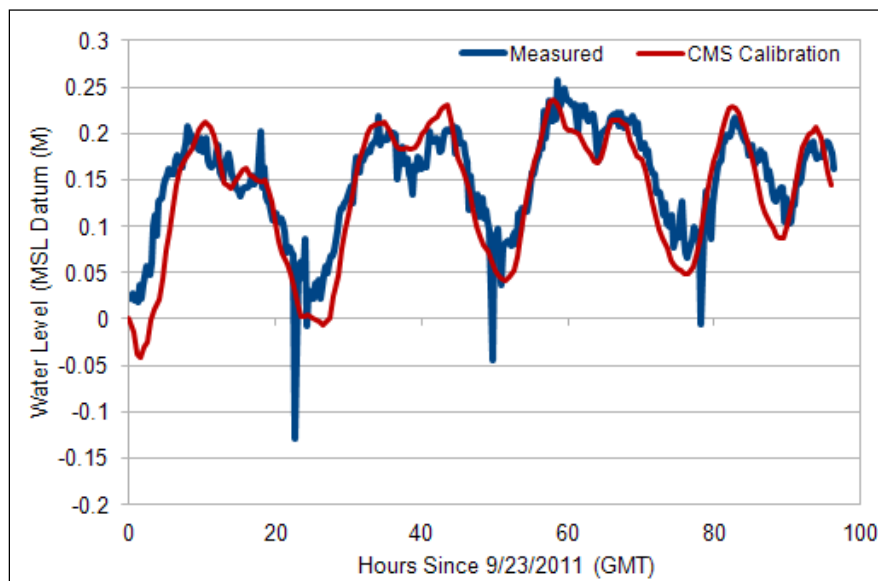
sensitivity testing, the model was tested with varying inflows from the San Bernard River. Although the inflows from this event exceeded 2,000 cubic feet/second (cfs) at the Boling, Texas, gauge on February 23, 2012, the inflows at the intersection with the GIWW were likely higher. Model results from a validation period run with a 1,900 cfs inflow and similar tide stage to when this photograph was taken show the observed flow pattern (Figure B11). The flow originating from Brazos River is a different color than the flow from the San Bernard River.

Figure B11. Left – high water event at San Bernard and Brazos Rivers (Friendsofsanbernard.org, Tom Taylor photographer); Right - October validation run with 1900 cfs peak discharge.
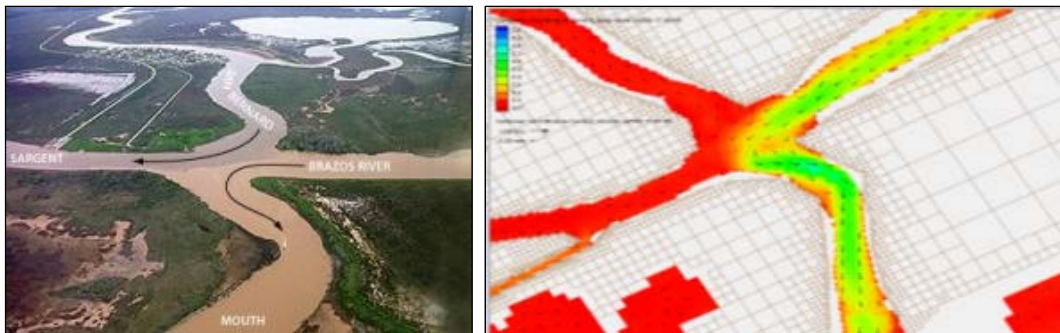
The next figure includes imagery taken from Google Earth dated February 16, 2010. There is a distinct line between the lighter-colored water on the Sargent, Texas, side of Mitchell's Cut compared to darker water on the Matagorda, Texas, side of Mitchell's Cut. The flow shown in the photograph is reproduced with the model (Figure B12).

Validation

Once the model is calibrated, then it can be validated against a different data set. The CMS was calibrated against data obtained from the installed tide gauge for the time period October 1-30, 2011. A detailed analysis of this validation dataset is provided.

Validation period forcing

Data from the same sources as discussed in the calibration section were used. As mentioned earlier, wind forcing tends to dominate water levels in Texas bays. Winds for this time period from the Brazos River flood gates and Port O'Connor are shown to illustrate the wind variability (Figures B13 and B14). Creating a spatially varying wind field for the model domain is not within the scope of this study. The wind shifts between north and south present a challenge to model water levels accurately in the model domain. Figure B15 shows the measured river discharge values used in the model for validation. Finally, Figure B16 shows the tide-water level data that was used for validation input.

Sensitivity

Flow measurements were not available for this study, and therefore validation is outside the scope of this study. To mitigate for this, a series of sensitivity tests were conducted to provide a measure of certainty in the model's capability to capture the relevant hydrodynamics of the system.

Figure B12. Google Earth imagery of Mitchell's Cut (top), Mitchell's Cut flow pattern with
computed current magnitude (bottom).
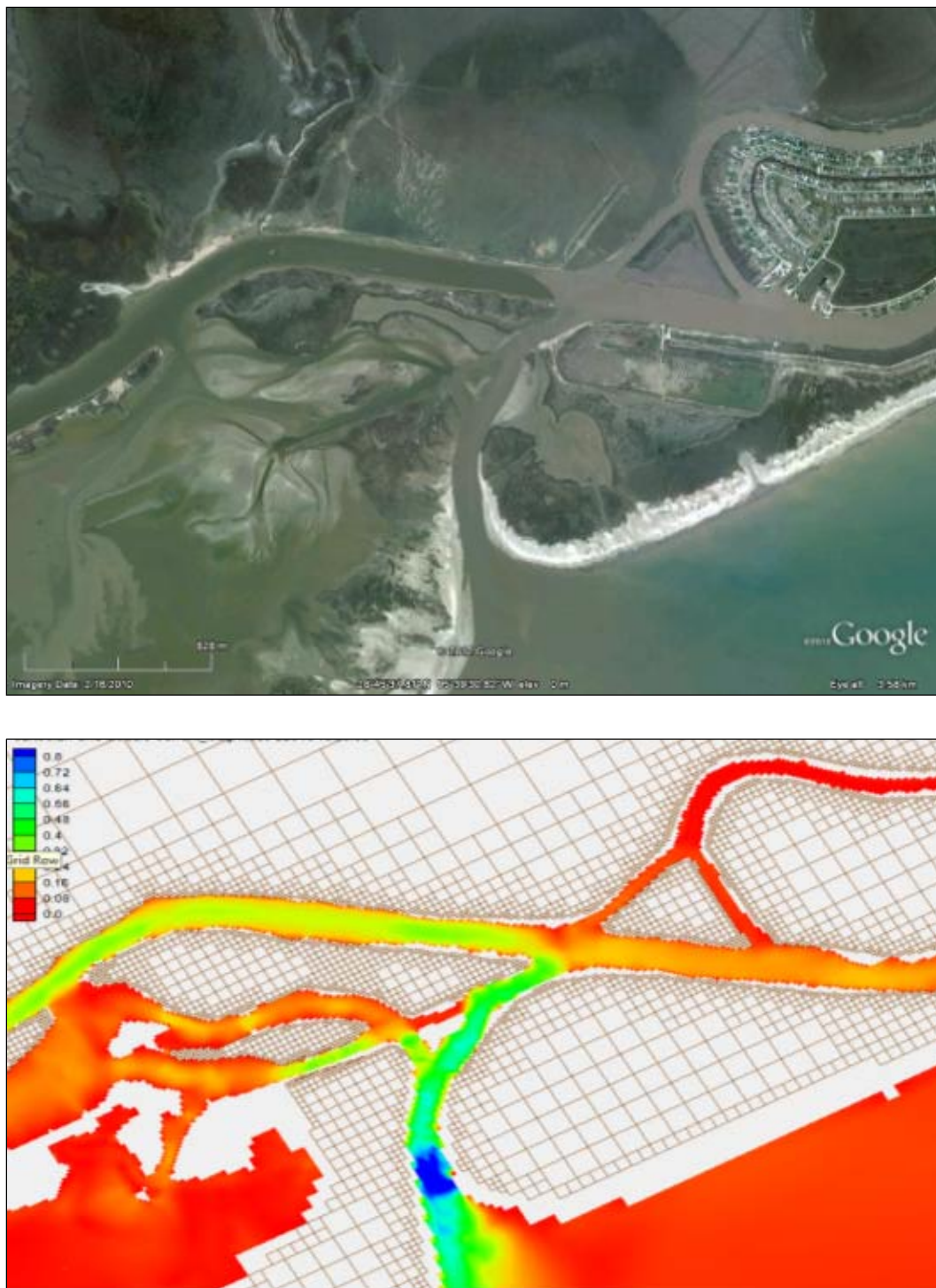
Figure B13. Validation period winds from the Brazos River flood gates.
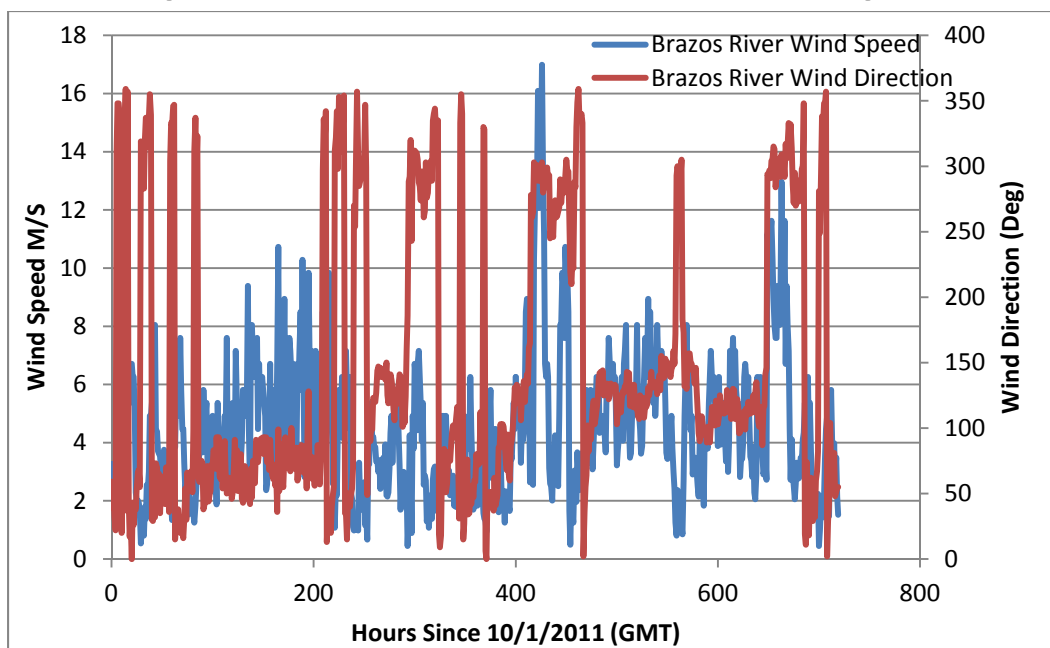
Figure B14. Validation winds from Port O'Connor.
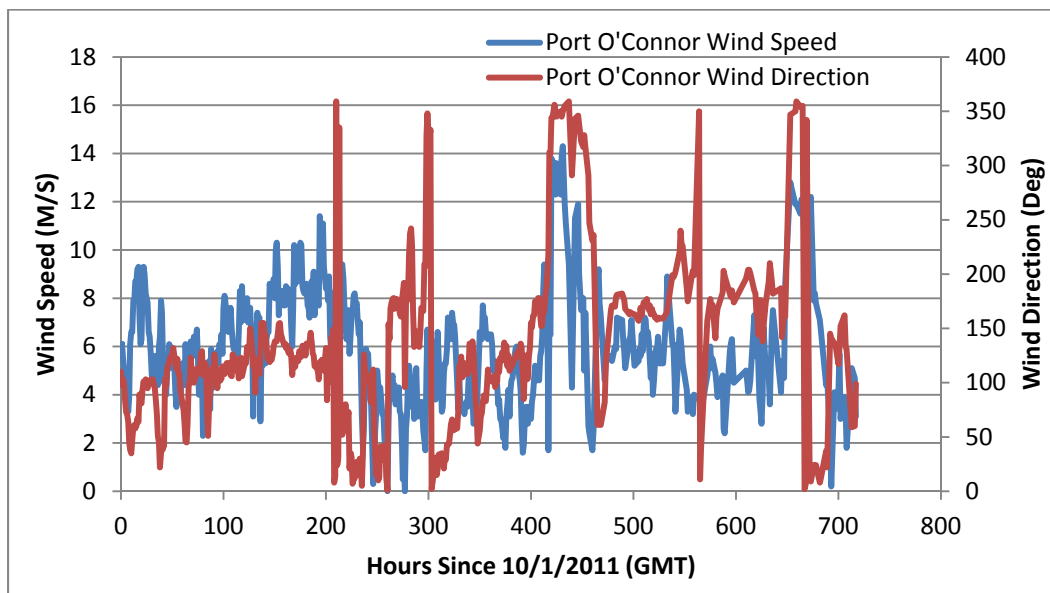
Figure B15. San Bernard and Brazos River discharge.
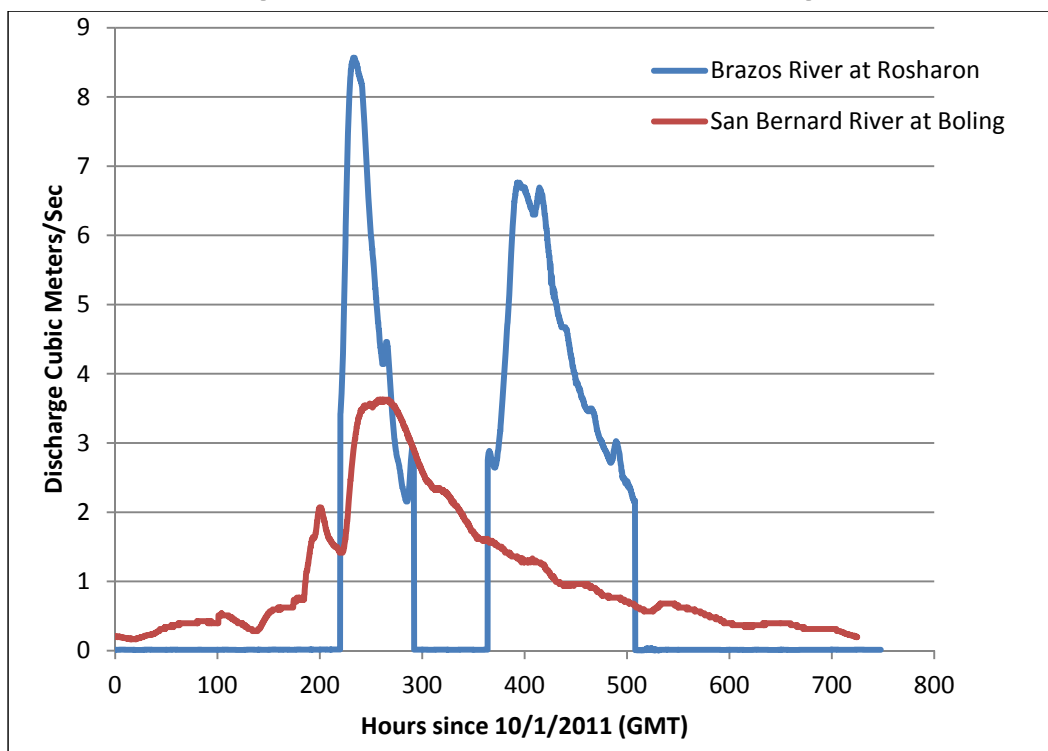
Figure B16. Measured water levels from Sargent, Texas, compared to the ocean forcing from Galveston pleasure pier.
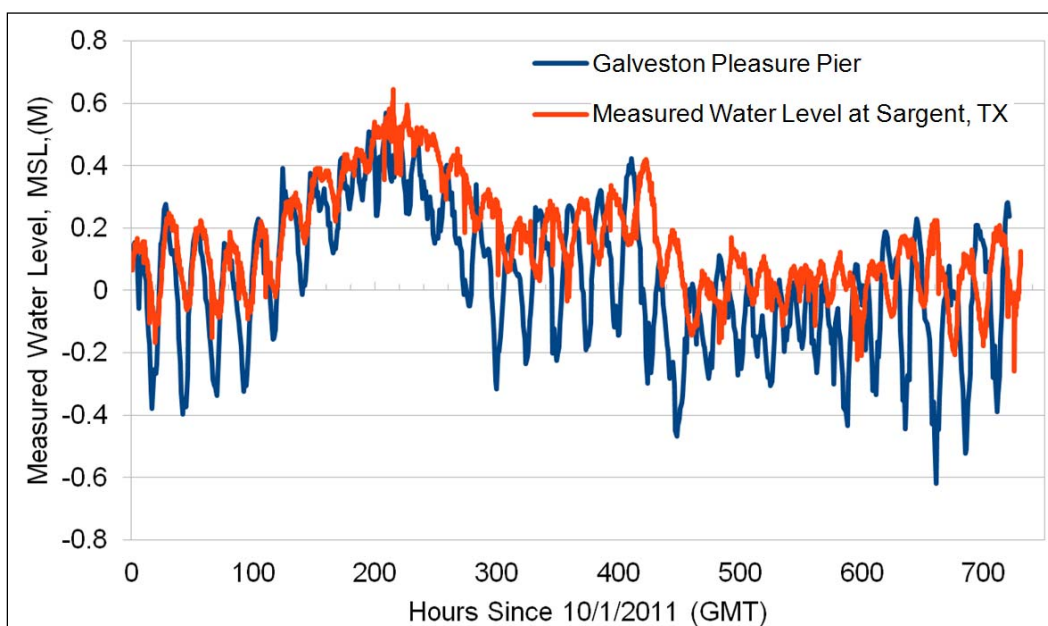
Storage

As noted in Sanchez and Parchure (2001), the surrounding intertidal wetlands and lakes present a significant modeling challenge. McNeal

Bayou, McNeal Lake, Pelican Lake, Redfish Bayou, Jones Lake, Cowtrap Lake, Cedar Lakes Creek, and Lake Austin all present a source of inflow and outflow to the GIWW. The bathymetry of these areas is not well known, but as per Sanchez and Parchure (2001), depths of 1 t0 1.5 m MSL were used.

Model results tend to have a greater deviation from measured values for north-to-south wind changes than for east-to-west variations. Therefore, varying amounts of storage were introduced into the model to simulate the storage effects of these shallow lakes and wetlands. The increased storage had little effect on the phase of the computed water levels but did increase their magnitude. CMS has some capability to simulate flow over vegetated ground, but no attempt was made to assign a percentage of the storage to wetlands and was instead assigned to the shallow lakes.

Grid extent

To avoid dependence on the Colorado River measured data and the datum adjustment to mean low tide datum, the model grid was modified to incorporate the entire Matagorda Bay based on previous work (Rosati et al. 2011) optimized for improved water level computation. Colorado River discharge from the Bay City gauge was added to model, and the flood gates were modeled as open. The grid also has the Brazos River and west lock represented so only external tide forcing and river inflows were used in the extended model grid (Figure B17). In this way, cause and effect on water levels and currents can be separated. Figure B18 shows that there are negligible differences between runs that use the CMS computed water levels or measured data for the Colorado River water levels.

Bathymetry

Water depths are not available from recent surveys for several locations in the model domain, East Matagorda Bay for example. Varying some of the depths in East Matagorda Bay from 1.2 to 2 m had no noticeable effect on these forcing data. Closing the San Bernard River mouth in the model had no noticeable effect on the model-data comparison. Closing the Brazos west flood gate either by itself or in conjunction with ignoring Colorado River and bypass influence did have a noticeable effect on the magnitude of the observed water levels, but the observed phase still followed the incident tidal forcing. Modifying the GIWW depths +/- 0.5 m and the mouth of Mitchell's Cut had no appreciable difference in model results as well.

Figure B17. CMS extended grid - overall grid, water elevations and wind vectors shown.
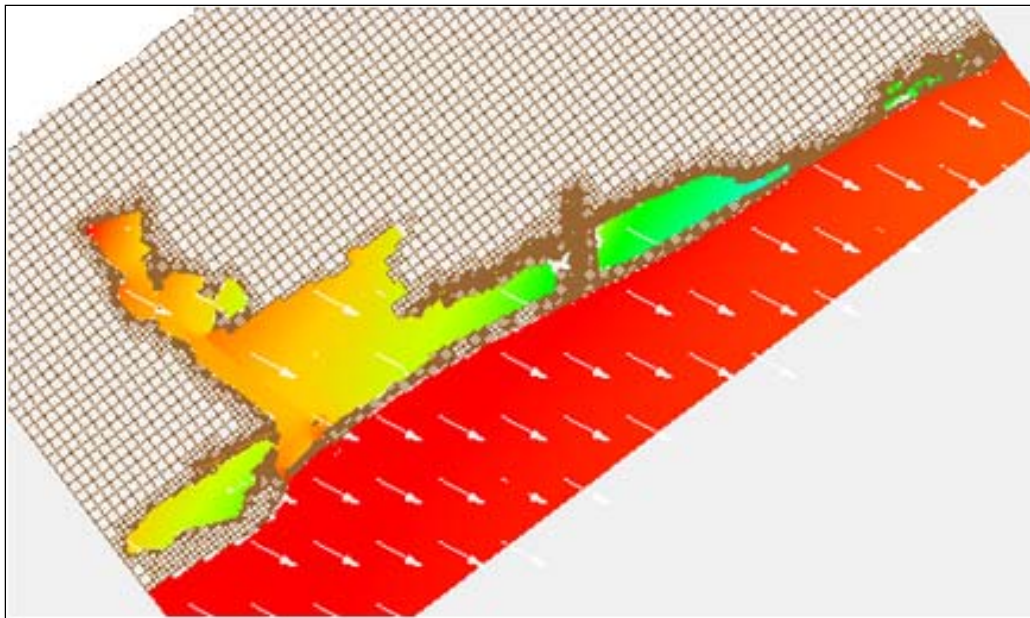
Figure B18. Comparison with Matagorda Bay vs. measured data.
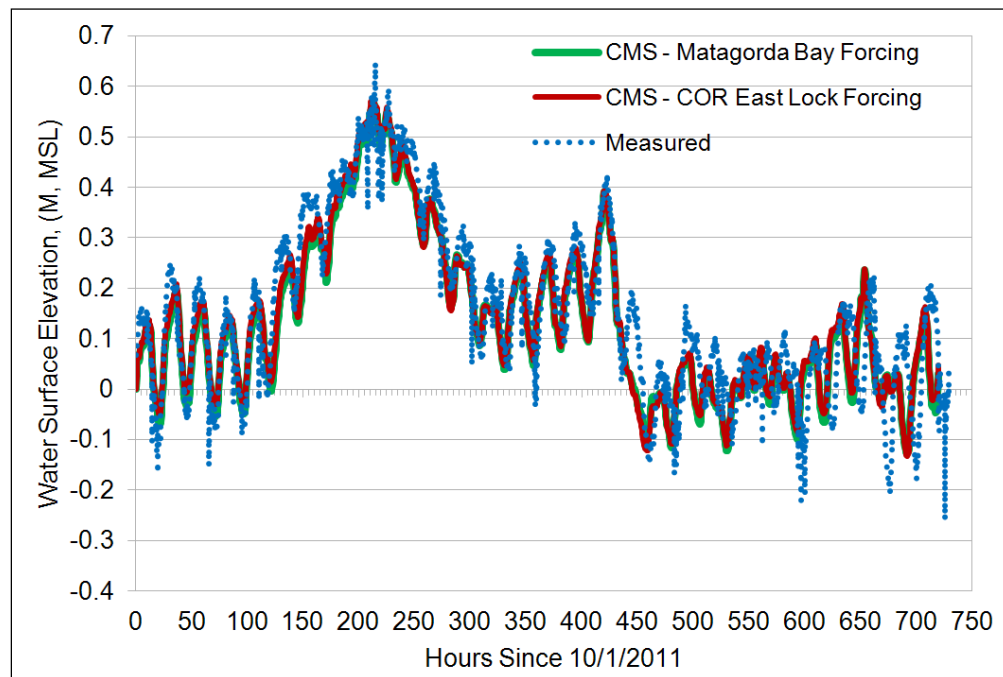
Winds

Figure B19 shows the results from two CMS model runs where only the uniform wind input forcing was changed between the winds presented in Figures B13 and B14. As expected, the winds have the greatest effect on water levels when the speeds are greatest. Note the significant difference

in water level during the time period between 400 and 450 hrs, where the wind velocity and especially direction persisted for a longer period for the Brazos River winds than the Port O'Connor winds.

Figure B19. Computed water levels based on different forcing conditions compared to measured data.
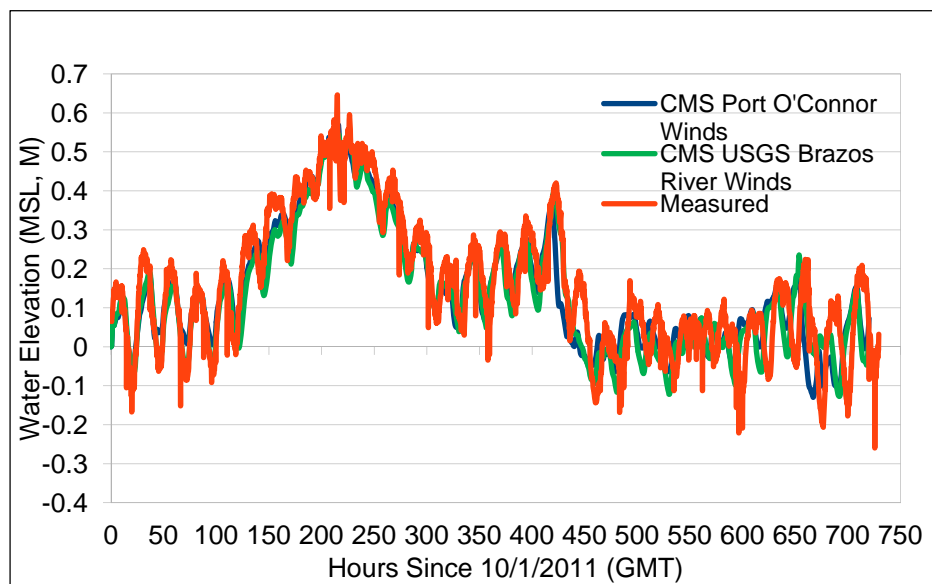
Validation and sensitivity discussion

The computed and measured results generally compare fairly well in magnitude; the computed results differ by 0.2 m at the time period centered around 450 hr into the simulation (Figure B20). From Figures B13 and B14, this is a time of high winds. Interestingly, Figure B19 shows the tide level outside Mitchell's Cut decreasing, while during this time period, the water level increased at the gauge. This relative increase in water level at this time is also observed at the Brazos and Colorado River USGS gauges. The next figure (Figure B20) shows the computed water level over most of the modeled domain with uniform wind vectors superimposed. Water levels equal to the measured values (0.2 m MSL) are seen in East Matagorda Bay, but not to the east of Mitchell's Cut. For the remainder of the simulation between 450 and 700 hr, there are at least two additional north-south wind shifts which produce a persistent 3-8 hr phase lag between the computed and observed water level values. The difference between the computed and observed values remains less than .1 of a meter, however.

Figure B20. Water levels at 450 hr into validation period simulation.
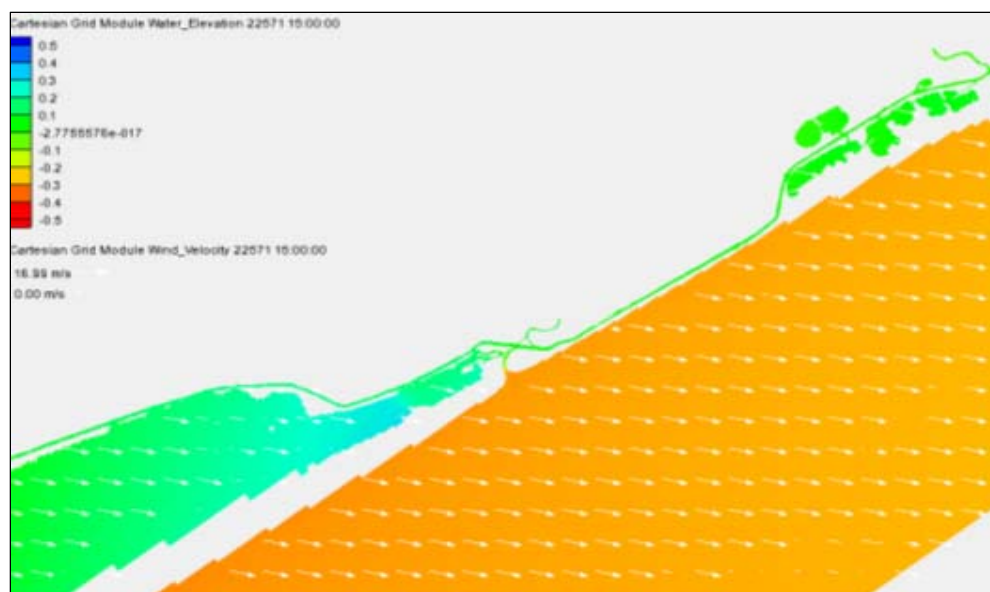
The relative insensitivity to most factors shows the expected dominance of the wind forcing on water levels. Including spatially varying winds should improve the model-data comparison (Table 6); however, wind measurement sites are limited, and the spatial resolution of model hindcast winds (0.25 deg) might not be sufficient for this application. Table B2 shows a summary of the sensitivity tests.

Table B2. Summary of sensitivity tests.

Description	Result
Modify GIWW depths +/- 0.5 m	Not significant
Modify Col. River levels by +/- 0.5 m	Significant, directly affects measured water levels near Mitchell's Cut
Vary Brazos River water levels used for forcing. +/- 0.5 m	Significant, affects measured water levels near Mitchell's Cut
Modify Manning's N flow resistance in the model between 0.016 and 0.02	No differences over 0.01 m were observed between model runs
Close San Bernard River mouth	No significant change in Mitchell's Cut water levels
Compare Port O'Connor, Brazos River floodgate winds	Significant. Largest differences during high wind periods
Vary storage represented by lakes and wetlands	Increased volume represented by increased water levels, however phase changes remained consistent with other results
Increase Brazos, San Bernard River inflows by factors of 10 and 16	Computed water levels and flow patterns have elevated water levels but the same phase

Sedimentation

CMS was run with waves and sediment transport enabled for the calibration data set. A sediment transport size of 0.2 mm was used as well as 0.14 mm (Appendix C). The sediment transport modeling approach was to use the model to determine qualitatively where sedimentation processes are most active and to delineate differences in sedimentation due to breakwater project effects. Figure B21 shows essentially no net erosion or deposition, but that the sediment is actively transported where the higher currents exist, as expected. Dredging in the GIWW does occur in the active section indicated by the model. The model results indicate that the GIWW can act as a sediment sink over longer periods of months rather than days at Mitchell's Cut.

Figure B21. CMS sedimentation results.
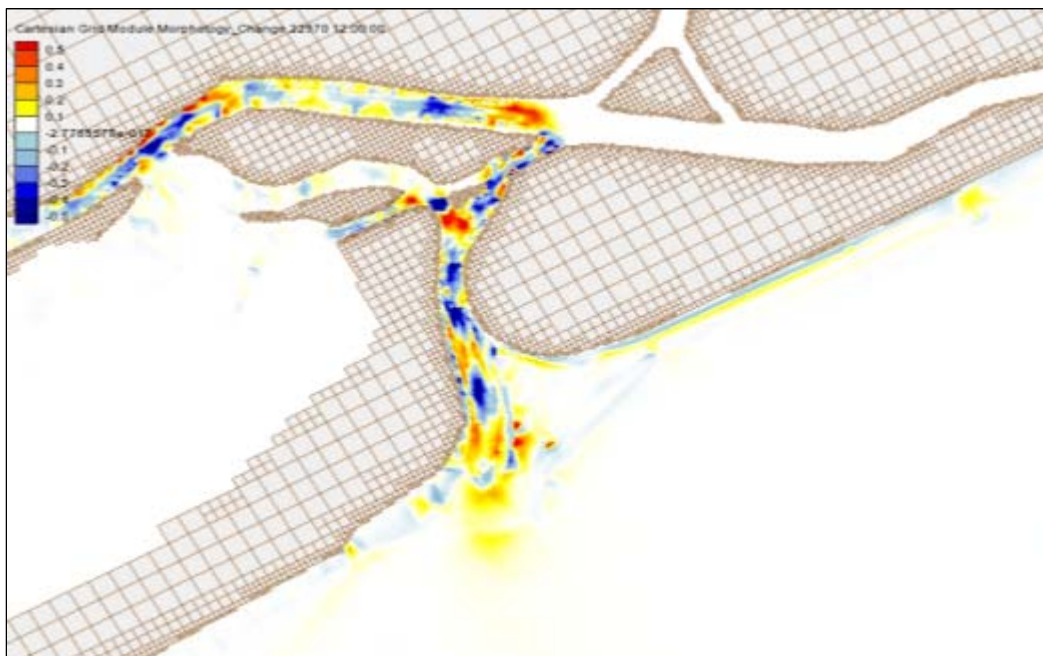
CMS has a mechanism to specify the transport grain size while varying the sediment D_{50} according to location. In Figure 10, a D_{50} of 2.0 mm was specified within Mitchell's Cut with a transport grain size of 0.14 mm. Besides the available surveys, no sedimentation data exist for model calibration/validation. A comparison of the 2010 (limited data available) and 2011 surveys (Figure B22) indicates some westward migration of the main inlet channel.

Figure B22. Survey data comparison between September 2011 and x
2010 surveys. Red = accretion, blue = erosion.

Wave breaking

The aerial photographs in Figure B23 and in Figure 28 help to describe the
wave breaking pattern at Sargent Beach. In Figure 28, the distance between
the two sandbars is approximately 500 ft for the western transect. The
500 ft roughly corresponds to the distance between the first intermittent
offshore breaker line and the breaker line closest to shore. The width of the
CMS-computed longshore current (Figure 47) approximates the distance
(150 ft) between the middle breaker line and the shoreline. Therefore, the
observed and computed wave patterns are consistent and are relevant to
alongshore transport in the area of interest.

Figure B23. Sargent Beach wave breaking pattern. Aerial photo taken July 14, 2011.

Appendix C: Sand Sample Analysis

Sand samples were collected along 12 of the 30 survey transects measures
on November 4, 2011. Samples were gathered by hand under water between
the berm and first bar. Table C1 lists the percent of sands, silts and clays
measured in each sample. Figures C1 and C2 respectively plot the distribu-
tion of sediments at Matagorda Peninsula and Sargent Beach. Figure C3
shows the location of transects where samples were collected along
Matagorda Peninsula (labeled as MP #). Figure C4 shows the location of
transects where samples were collected along Sargent Beach (labeled as
SB #).

Table C1. Distribution of sediment collected at Sargent Beach and Matagorda Peninsula.

Sample	Percentage											
	Sand	Silt	Clay	Very fine silt	Fine Silt	Med Silt	Coarse Silt	Very Fine Sand	Fine Sand	Med Sand	Coarse Sand	Very Coarse Sand
SB 4	99.96	0.04	0.0	0.0	0.0	0.0	0.04	31.45	64.09	4.42	0.0	0.0
SB 6	99.88	0.12	0.0	0.0	0.0	0.0	0.12	34.11	61.63	4.13	0.0	0.0
SB 8	99.83	0.17	0.0	0.0	0.0	0.0	0.17	39.15	58.66	2.01	0.0	0.0
SB 10	100.0	0.0	0.0	0.0	0.0	0.0	0.0	26.49	67.22	6.29	0.0	0.0
SB 12	99.89	0.12	0.0	0.0	0.0	0.0	0.12	38.52	59.73	1.64	0.0	0.0
SB 14	99.86	0.14	0.0	0.0	0.0	0.0	0.14	38.89	59.34	1.63	0.0	0.0
SB 16	99.94	0.06	0.0	0.0	0.0	0.0	0.06	36.27	61.72	1.95	0.0	0.0
SB 18	99.91	0.09	0.0	0.0	0.0	0.0	0.09	45.63	54.08	0.20	0.0	0.0
SB 20	99.85	0.15	0.0	0.0	0.0	0.0	0.15	42.10	56.91	0.84	0.0	0.0
SB 21	99.85	0.15	0.0	0.0	0.0	0.0	0.15	41.13	57.64	1.08	0.0	0.0
MP 1	100.0	0.0	0.0	0.0	0.0	0.0	0.0	16.62	66.74	16.65	0.0	0.0
MP 9	100.0	0.0	0.0	0.0	0.0	0.0	0.0	13.72	65.16	21.12	0.0	0.0

Figure C1. Grain size of sediment at Matagorda Peninsula.

Figure C2. Grain size of sediment at Sargent Beach.

Figure C4. Sargent Beach sediment samples.
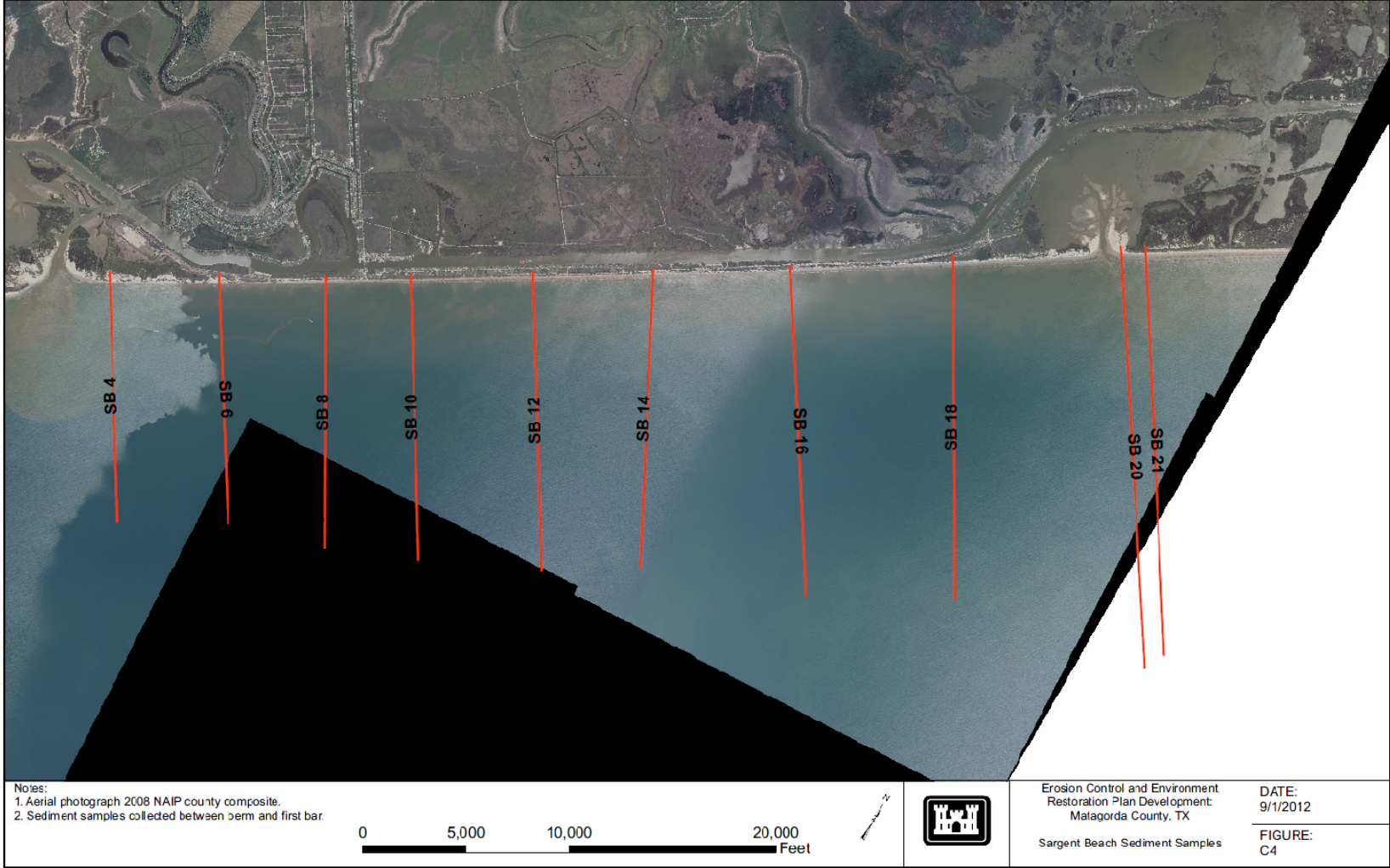
Appendix D: Plan Sheets

Figure D1. Matagorda groin system plan.
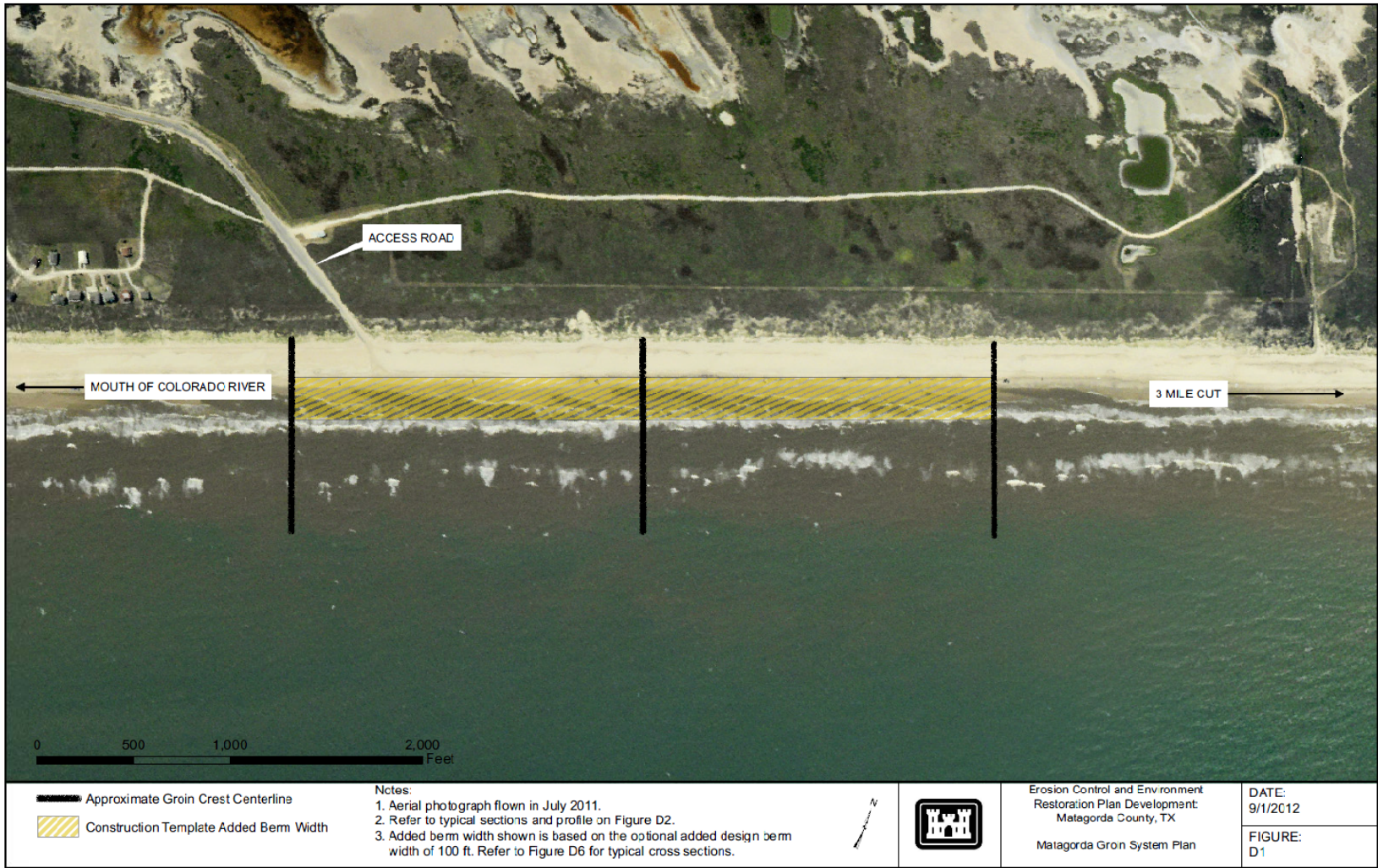
Figure D2. Matagorda groin profile and cross section.

TYPICAL GROIN PROFILE

NOT TO SCALE

TYPICAL GROIN CROSS SECTION

NOT TO SCALE

Notes:
1. These preliminary graphics are not intended to be used for construction, bidding, or permitting.
2. Dimensions shown are approximate.
3. Excavate to -6.7 FT NAVD on landward section and -10.0 FT NAVD on seaward section.
4. Refer to text for preliminary layer thickness and other requirements.

Erosion Control and Environment
Restoration Plan Development:
Matagorda County, TX

Matagorda Groin
Profile and Cross Section

DATE:
9/1/2012

FIGURE:
D2

Figure D3. Sargent Beach Phase 1 plan.

Figure D4. Sargent Beach breakwater typical plan.

TYPICAL EXISTING SHORELINE

5 CY/LF BEACH FILL
CONSTRUCTION TEMPLATE BERM WIDTH

APPROX
350'

220'   330'   220'

BREAKWATERS

TYPICAL BREAKWATER PLAN
NOT TO SCALE

Notes:
1. These preliminary graphics are not intended to be used for construction, bidding, or permitting.
2. Dimensions shown are approximate.
3. Refer to Figure D5 for breakwater cross section and profile.
4. Refer to Figure D6 for beach fill typical cross section.

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Matagorda County, TX

Sargent Beach Breakwater Typical Plan

DATE:
9/1/2012

FIGURE:
D4

Figure D5. Sargent Beach breakwater cross section and profile.

-4.0' NAVD

220'

2000# - 6000#
GRADED RIPRAP
MIN THICKNESS: 6'

3
1

3
1

2# - 25# BEDDING STONE
MIN THICKNESS: 1.2'

GEOGRID COMPOSITE FILTER

APPROX. EXISTING
GRADE (VARIES)

APPROX. EXISTING
GRADE (VARIES)

TYPICAL BREAKWATER PROFILE
NOT TO SCALE

+4.0' NAVD

10'

APPROX. EXISTING
GRADE (VARIES)

3
1

2000# - 6000#
GRADED RIPRAP
MIN THICKNESS: 6'

3
1

GEOGRID COMPOSITE FILTER

2# - 25# BEDDING STONE
MIN THICKNESS: 1.2'

APPROX. EXISTING
GRADE (VARIES)

TYPICAL BREAKWATER CROSS SECTION
NOT TO SCALE

Notes:
1. These preliminary graphics are not intended to be used for construction, bidding, or permitting.
2. Dimensions shown are approximate.
3. Excavate if required to meet the minimum bedding and GRR layer thicknesses.
4. Typical elevation of existing grade at breakwater centerline is -7.0 FT NAVD (varies).

Erosion Control and Environment
Restoration Plan Development:
Matagorda County, TX

Sargent Beach Breakwater
Cross Section and Profile

DATE:
9/1/2012

FIGURE:
D5

Figure D6. Typical Beach construction templates.

TYPICAL SECTION: 200 FT
BEACH FILL CONSTRUCTION TEMPLATE

379 FT
4.0 FT NAVD
175 CY/LF
30
1
-10.7 FT NAVD
APPROX. EXISTING BEACH PROFILE (VARIES)

TYPICAL SECTION: 50 FT
BEACH FILL CONSTRUCTION TEMPLATE

125 FT
4.0 FT NAVD
44 CY/LF
30
1
-9.1 FT NAVD
APPROX. EXISTING BEACH PROFILE (VARIES)

TYPICAL SECTION: 100 FT
BEACH FILL CONSTRUCTION TEMPLATE

211 FT
4.0 FT NAVD
88 CY/LF
30
1
-9.7 FT NAVD
APPROX. EXISTING BEACH PROFILE (VARIES)

TYPICAL SECTION: 5 CY/LF
BEACH FILL CONSTRUCTION TEMPLATE

37 FT
4.0 FT NAVD
5 CY/LF
-1.1 FT NAVD
APPROX. EXISTING BEACH PROFILE (VARIES)

Notes:
1. These preliminary graphics are not intended to be used for construction, bidding, or permitting.
2. Dimensions shown are approximate.
3. Typical existing beach profile shown is based on merging and smoothing adjacent profiles.
4. Construction template is shown. Natural processes will distribute the sediment along the profile over a few months, reducing the dry beach width to the design width, the value shown in the section labels.

Erosion Control and Environment Restoration Plan Development: Matagorda County, TX

Typical Beach Construction Templates

DATE: 9/14/2012

FIGURE: D6

REPORT DOCUMENTATION PAGE

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14. ABSTRACT

A two-part study was conducted to identify structural methods to reduce beach erosion in Matagorda County at Sargent Beach and along Matagorda Peninsula east of the Mouth of the Colorado River (MCR), Texas. Phase 1 (Thomas and Dunkin 2012) investigated the coastal processes of the region and introduced several structural alternatives to reduce erosion at both locations. Preferred alternatives included a groin field with beach fill east of MCR and segmented breakwaters to protect Sargent Beach.

This report (Phase 2) further details coastal processes with a short-term, recent sediment budget, describes recent studies, provides higher resolution numerical modeling, and includes preliminary design of the selected alternatives. Numerical models were applied to evaluate structure performance and inform design. GenCade calibration was updated based on model improvements and additional data. A Coastal Modeling System (CMS) numerical model was developed and validated against water level data.

15. SUBJECT TERMS Beach erosion Breakwaters	Coastal modeling system Coastal processes GenCade	Groins Sediment budget Shoreline change

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